Drainage Design Manual

for Maricopa County, Arizona

2801 West Durango Street, Phoenix, Arizona 85009 (602) 506-1501

Adopting Agencies

The following agencies and municipalities adopt the methodologies incorporated into the *Drainage Design Manual for Maricopa County, Hydrology*.

Adopted By:	Date
Flood Control District of Maricopa County	
City of Phoenix	
Maricopa County Department of Transportation	

Acknowledgements

The information, procedures, and recommendations that are presented in this manual are mainly the result of previously published efforts of many diligent and talented engineers and scientists. The authors of this manual have made every effort to cite the original authors and researchers whose contributions to this manual, and to the science of hydrology, are greatly appreciated.

The authors of this manual are indebted to the many individuals and organizations, including the staff at the Flood Control District, that have supported this effort through recommendations, technical guidance, encouragement, and review of draft sections of this manual.

The first three editions (1990, 1992 & 1995) of this manual were authored by:

George V. Sabol, PhD, PE, Consulting Engineer

J.M. Rumann, Hydrologist

Davar Khalili, PhD, Hydrologist

Stephen D. Waters, Hydrologist

Ted Lehman, Hydrologist

Engineering Division
Flood Control District
of Maricopa County

The fourth edition of the manual was updated in a collaborative effort between the Flood Control District and the City of Phoenix. Stantec Consulting Inc. facilitated the updating of the manual. The Flood Control District provided recommendations and review comments. The major contributors to the fourth edition are:

R. Michael Gerlach, PE, Consulting Engineer

Amir M. Motamedi, PE, Hydrologist

Thomas R. Loomis, PE, RLS, Hydrologist

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

AGENDA FORM

Contract/Lease for ☐ NEW ☐ RENEWAL ☐ AMENDMENT ☐ CANCELLATION
(for existing, record Encumbrance No. below)

LOW ORG. NO. 6900 DEPARTMENT: Flood Control District CONTROL NUMBER: FCD-1241

ENCUMBRANCE NO. ______ AGENCY: Public Works CONTROL NUMBER: PW-1241

1. BRIEF DESCRIPTION OF PROPOSAL AND REQUESTED BOARD ACTION: It is requested that the Board of Directors approve a resolution adopting volume one of a two-volume drainage design manual. Entitled the Hydrologic Design Manual for Maricopa County, volume one provides technical procedures for estimating stormwater runoff to assist engineers in the design of storm drainage facilities. Volume two of the drainage design manual will provide "hydraulic" design guidelines as opposed to "hydrologic" procedures and will be presented for the Board's consideration at a future date. Development of the manuals was among the objectives of a multi-jurisdictional task force formed by the District in 1985 to establish a common basis for drainage management within Maricopa County. By formally adopting volume one, the Board will establish the hydrologic design procedures described in the manual for use by District staff, by jurisdictions cost-sharing with the District in flood control projects, by contractors performing work for the District and, beginning January 1, 1992, by all parties submitting drainage reports and studies to the District for review and approval. The Flood Control Advisory Board recommended adoption of this resolution at its January 23, 1991 meeting.

2. COMPLIANCE WITH MARICOPA COUNTY PROCUREMENT CODE N/A (article) N/A (paragraph) Procurement Officer

SOLE SOURCE JUSTIFICATION ______

3. CONTINUED FROM MEETING OF ______
DISCUSSED IN MEETING OF ______

4. ☐ **THIS DEPARTMENT WILL CAUSE PUBLICATION**
☐ **CLERK OF THE BOARD TO CAUSE PUBLICATION**

5. MOTION: It is moved that the Flood Control District of Maricopa County Board of Directors . . adopt Resolution FCD 91-03, the Hydrologic Design Manual for Maricopa County, thereby requiring its use by jurisdictions cost-sharing with the District in flood control projects, by contractors working for the District, and beginning January 1, 1992, by all parties submitting drainage reports and studies to the District for review and approval.

6. FINANCIAL: ☐ Expenditure ☐ Revenue ☐ Budgeted ☐ Contingency ☐ Budget Amendment ☐ Transfer ☐ Grant or other
Adopt Resolution

$ 0 (Total) N/A (Fund) Financial Officer 4-2-91 (Date)

7. PERSONNEL:
Personnel Director ______ Date ______

8. FLOOD CONTROL DISTRICT:
Action Recommended by 3-26-91 (Date)

9. MATERIALS MANAGEMENT:
A. Materials Management Director ______ Date ______
B. W/MBE Representative ______ Date ______

10. LEGAL: Approved as to form and within the powers and authority granted under the laws of the state of Arizona to the Flood Control District of Maricopa County Board of Directors.
General Counsel 3-27-91 (Date)

11. OTHER: ______
Signature ______ Date ______

12. APPROVED FOR AGENDA:
Approving Official 3-28-91 (Date)

13. OTHER: ______
Signature ______ Date ______

15. RECOMMENDATION OF COUNTY MANAGER:
☒ Approve ☐ Disapprove
Comments:
Roy R. Pederson
County Manager ______ Date ______

14. BOARD OF DIRECTORS: Action taken:
☒ Approved ☐ Amended ☐ Disapproved ☐ Deleted
Continued to: (Date and type of meeting)
Clerk of the Board APR 15 1991 (Date)

6910-001 R5-90

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Comments

Users of this manual are strongly encouraged to submit any comments, criticisms, or findings of errors. This information should be addressed to:

Engineering Division Manager
Flood Control District of Maricopa County
2801 West Durango
Phoenix, AZ 85009

Because of ongoing legal and technical changes in the field of stormwater management, revisions to this manual will be required from time to time. Such revisions will take place on an ongoing, as needed basis and will be posted on the FCDMC's Web page (www.fcd.maricopa.gov). A separate document available on the FCDMC's Web page will summarize revisions made after the release of this third edition.

Revisions

Because of ongoing technical and administrative changes in the field of stormwater management, revisions to this manual will be required from time to time. Such revisions will take place on an ongoing, as needed, basis and will be posted on the FCDMC's Web page (www.fcd.maricopa.gov). The dates of revision and an overview of changes made are listed below.

1st Edition	September 1, 1990
2nd Edition	June 1, 1992
3rd Edition	January 1, 1995
4th Edition	September 2003 (draft)

Overview of Changes Made in the Second Edition

Title - The title of the document has changed. The hydrology and hydraulics manuals are now the Drainage Design Manual for Maricopa County, Volumes I and II, respectively.

Adoption - A copy of the Agenda Form, signed by the Board of Directors on April 15, 1991, is included. This form indicates formal adoption of the manual, requiring it use by jurisidictions that cost-share with the District in flood control projects, by contractors working for the District, and by all parties submitting drainage reports and studies to the District for review and approval.

Document Page Numbering - Page numbering has changed to section numbering rather than consecutive (ie., 1-1, 2-1, 3-1, etc.).

Chapter 2 - The rainfall chapter has been substantially condensed. The computer program PREFRE has been added to ease development of rainfall statistics for sites outside the Phoenix metropolitan area. The PREFRE user's manual is included with the manual as Appendix J. An additional isopluvial map with 2-hour, 100-year depths has been added.

Chapter 3 - New roughness factor descriptions were developed. '/C" coefficients will now be adjusted to reflect storm frequency, and a new table is included. A computer program RAnoNAL.EXE is included for development of discharges and volumes using the Rational Method.

Chapter 4 - The methodology used to develop Green and Ampt loss parameters has been substantially modified and simplified. The section on the Initial plus Uniform Loss Rate Method has been reduced, and limitations for the use of that method are provided. An equation is provided for calculation of the XKSAT vegetation adjustment coefficient.

Chapter 5 - New land classification descriptions are provided to facilitate selection of parameters in the Kb equation. An error was corrected in the Lag equation (the Corps of Engineers uses $C = 24K_n$ instead of $C = 20K_n$). The MCUHPl and MCUHP2 computer programs were revised to reflect our change our address, some additional data inputs were added to facilitate revisions and an error was corrected in the 2-hour storm distribution (the program was underestimating T_c because of an incorrect summation of the first three rainfall excess values).

Chapter 6 - The routing chapter now includes guidance on using the Muskingum-Cunge routing option recently available in HEC-l. A sample problem is included in the Examples section.

Chapter 7, the Appendices, and the Examples - All have been updated to incorporate the changes outlined above.

Overview of Changes Made in the Third Edition

In addition to the correction of a few typographical errors, changes of January 1, 1995 revision of the Drainage Design Manual, Volume I, Hydrology included the following:

Chapter 2 -The SCS Type II rainfall distribution is recommended for use for the 24-hour general design stonn. Areal reductions of point rainfall are to be made with Table 2.1 a, which is based on the NWS-HYDRO 40 data. Guidelines have also been added as to when to select the general storm for use in design hydrology in Maricopa County.

Chapter 3 - The RATIONAL.EXE program has been updated to better match 10-year rainfall intensities for durations between 10 and 20 minutes as shown on the I-D-F curve, Figure 3.2. The revised program is supplied on the DDMS diskette available with this revision (see 6. below).

Chapter 4 - A table has been added to help with the selection of IA, RTIMP, and percent vegetation cover for representative urban land use types in Maricopa County.

Chapter 5 - Two new S-graphs have been added for use in Maricopa County. The newly added S-graphs are the Desert/Rangeland S-graph and the Agricultural S-graph. A table has also been added to facilitate the selection of S-graph type and K_n values for those S-graphs for estimation of basin lag time.

Chapter 6 - The Normal-Depth routing method has been added to the Manual as an additional routing method for use in flood hydrology studies in Maricopa County.

Appendix I - A new computer program and user's guide have been added to this revision of the Manual. The new program brings together the PREFRE program, a modified version of the loss parameter spreadsheet functionality, and the MCUHP programs to speed up the creation of HEC-I models using the methodologies recommended in the Manual. Additionally, two changes have been made to the MCUHP programs. First, the SCS Type II 24-hour design storm temporal distribution has been corrected and is now entered into the HEC-I data file as a 15 minute distribution. Second, the two S-graphs added to Chapter 5 have been incorporated into the MCUHP2 program.

Appendix K - An appendix of K_n values for various real watersheds has been supplied for additional help in the selection of watershed K_n values. These data were taken from a report by George V. Sabol Consulting Engineers, Inc., performed for the District since the last Manual revision.

Overview of Changes Made For The Fourth Edition

All Chapters - Policies and standards were removed to a separate volume entitled *Policies and Standards for Maricopa County, Arizona,* 2003. This allows each jurisdictional entity to customize its policies and standards to meet its community's needs. Also all references to the MCUHP programs were changed to DDMSW.

Chapter 1 Introduction – In general, the contents were reformatted into a single section. Also, a brief discussion of the contents of each chapter was added.

Chapter 2 Rainfall – The table identifying design rainfall criteria is eliminated as this information is listed in the *Policies and Standards for Maricopa County, Arizona,* 2003. Procedures for determining the design rainfall criteria were expanded. The isopluvial figures were moved to Appendix A.

Chapter 3 Rational Method – The I-D-F graph was moved to Appendix B. A discussion of the computation of site specific intensities was added and is intended to replace the I-D-F graph.

Procedures for determination of peak discharge at multiple points in a drainage network was added.

Chapter 4 Rainfall Losses – Procedures for the determination of the rainfall loss variables of the Green and Ampt equation were expanded.

Chapter 5 Unit Hydrograph – Procedures for the determination of the Clark unit hydrograph parameters and the S-Graph ordinates were expanded.

Chapter 6 Multiple Frequency Modeling – This is an entirely new chapter.

Chapter 7 Channel Routing – The Channel Routing chapter was changed to Chapter 7. The contents of this chapter were reorganized.

Chapter 8 Indirect Methods – This is an entirely new chapter.

Chapter 9 Application – The Application chapter was changed to Chapter 9. The procedures presented in Chapters 2 through 8 were added. User notes regarding the procedures and application of the methodologies presented in this manual were added along with detailed examples specific to each chapter.

Fourth Edition Dates of Revisions

The following indicates the dates in which the fourth edition has been updated and summarizes revisions made after the release of this fourth edition.

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1 INTRODUCTION

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1.1 OVERVIEW

The objective of the *Drainage Design Manual for Maricopa County, Hydrology*, (hereinafter referred to as the *Hydrology Manual*) is to provide technical procedures for the estimation of flood discharges for the purpose of designing stormwater drainage facilities in Maricopa County. Two methodologies are defined for the development of design discharges; the Rational Method, and rainfall-runoff modeling using a design storm. For small, urban watersheds, less than 160 acres and fairly uniform land-use, the Rational Method is acceptable. Use of this method will only produce peak discharges and runoff volumes and this method should not be used if a complete runoff hydrograph is needed, such as for routing through detention facilities. For larger, more complex watersheds or drainage networks, a rainfall-runoff model should be developed. The *Hydrology Manual* provides guidance in the development of such a model and the estimation of the necessary input parameters to the model. Although not necessarily required, the use of the U.S. Army Corps of Engineers' HEC-1 Flood Hydrology Program facilitates the use of the procedures that are contained in the *Hydrology Manual*. (The *Hydrology Manual* was written to supplement the HEC-1 User's Manual.) The manual also provides indirect methods intended to be used as confidence checks and verification of the reasonableness of the results obtained from the two methodologies discussed above.

The *Hydrology Manual* can be used to develop design discharge magnitudes for storms of frequencies up to and including the 100-year event. The design storm is of 6- or 24-hour duration and that storm is to be used for the design of all stormwater drainage facilities except stormwater storage facilities. The criteria to be applied to the 2-hour storm is also provided in the *Hydrology Manual* for use in design of stormwater storage facilities, as a minimum recommended criteria for Maricopa County. The criteria for design of stormwater storage facilities in unincorporated areas of Maricopa County is the 100-year, 2-hour storm. Although this is the minimum recommended criteria for all of Maricopa County, the Policies and Standard manual for each jurisdictional entity should be referenced for specific guidance for incorporated areas.

The rainfall-runoff modeling procedure that is contained in the manual is physically based, that is, the procedures are based, to the extent practical, on the physical processes that occur during the generation of storm runoff from rainfall. While the basic procedure is physically based, this does not assure that the rigorous application of the procedures will, in fact, reproduce the actual rainfall-runoff phenomenon of any storm that has occurred or may occur in the future. However, the procedure, when applied with good hydrologic judgement, should yield consistent results for design purposes.

Throughout the development of the *Hydrology Manual* three benchmarks were continually applied in judging the applicability of individual procedures and the overall methodologies; **accuracy, practicality, and reproducibility**. Accuracy is a measure of how well the results of the procedure reproduce the physical process being simulated. Although accuracy is highly desired, it is theoretically impossible to achieve in an earth science such as hydrology, and in a practical sense, accuracy is not feasible to assess except for a few situations where adequate verification data are available. Relative accuracy was assessed throughout the development of the procedures in the manual through testing and verification against recorded data.

Practicality is a user's decision regarding the best and most appropriate level of technology to apply considering the information that is available, anticipated user, consequences of error, and desired or required output. Whereas both simpler procedures and more sophisticated procedures are available, the adopted methodologies provide a compromise between these two extremes, and the best practical level of technology is judged to be recommended in the manual considering the state of current hydrologic knowledge of arid and semi-arid lands.

Reproducibility is a characteristic that provides reasonable confidence that consistent results will be achieved by all qualified users. Reproducibility is highly desirable for a design standard in order to eliminate, to the extent possible, unnecessary conflicts over the interpretation and application of the design method. Reproducibility is achieved through clear and concise manual procedures and user guidance. Every effort has been made toward this end.

A brief discussion of the content of each chapter of the *Hydrology Manual* follows:

Chapter 1 ***Introduction*** - The introduction states the purpose, scope and limitations, and general use of the manual.

Chapter 2 ***Rainfall*** - The characteristics of severe storms in Maricopa County are documented as a setting for defining the design rainfall criteria. Procedures and information are provided for the determination of depth-duration-frequency statistics of storms in Maricopa County. These are derived from NOAA Atlas 2, Arizona, which is currently the most comprehensive and authoritative source of such information. The limitations and potential inaccuracy of the NOAA Atlas are recognized and until an equivalently accepted source of rainfall statistics is provided, this source must be used. Recent reanalysis of the short duration (less than 1-hour) rainfalls by the National

Oceanographic and Atmospheric Administration have been used as a supplement to the NOAA Atlas.

The temporal distribution of rainfall for the majority of design conditions is a 6-hour local storm. The 6-hour storm distribution is based on an analysis by the U.S. Army Corps of Engineers, Los Angeles District, of the 19 August 1954 Queen Creek storm. The Corps' distribution has been modified somewhat to reflect the design rainfall criteria that are desired for use in Maricopa County, and this modification includes using the hypothetical distribution for drainage areas less than 0.5 square miles. The temporal distribution is a function of drainage area and this is to reflect the spatial variability of rainfall intensities that are known to exist with severe local storms in Maricopa County. A 2-hour distribution is provided for use in the design of stormwater storage facilities. The reduction of rainfall depth with storm area for the 6-hour rainfall is accounted for by a depth-area reduction curve based on the 1954 Queen Creek storm. In some cases, a general storm may be the accepted design rainfall. In Maricopa County, the general storm to be used is the SCS Type II pattern using areal reductions of point rainfall.

Chapter 3 ***Rational Method*** - Use of the Rational Method is to be limited to an area of up to 160 acres. The watershed should be of uniform land use for application of this method. Intensity-duration-frequency (I-D-F) statistics are to be obtained from the information contained in Chapter 2. An equation for the estimation of time of concentration is provided which is a partial function of rainfall intensity. Values of the runoff coefficient “C” to be applied to various land uses in Maricopa County are provided.

Chapter 4 ***Rainfall Losses*** - The preferred method for the estimation of rainfall losses is the Green and Ampt infiltration equation with an estimate of surface retention loss. This requires the classification of soil according to soil texture, which is available for most of Maricopa County. Adjustment of the loss rate is available as a function of vegetation cover. Other methods are available to estimate rainfall losses if adequate soils and/or vegetation data are not available.

Chapter 5 ***Unit Hydrograph Procedures*** - The use of unit hydrographs to route rainfall excess from the land's surface is recommended and the procedures recommended to do so are either the Clark unit hydrograph or the application of selected S-graphs. The Clark unit hydrograph is recommended for watersheds or subbasins less than 5 square miles in size with an upper limit of application of 10 square miles. Procedures are provided for the estimation of the two numeric parameters: time of concentration and storage coefficient. Two default time-area relations are provided; one for urban watersheds and the other for natural watersheds. Four S-graphs have been selected for use in flood hydrology studies of major watercourses in Maricopa County. The Phoenix Mountain, Phoenix Valley, Desert/Rangeland, and the Agricultural S-graphs are described and guidelines are provided for their selection. A procedure is provided for the estimation of the S-graph parameter, lag.

Chapter 6 *Multiple Storm Frequency Modeling* - Runoff hydrographs for the 2-, 5- and 10-year events are to be estimated by the application of ratios to the 100-year runoff hydrograph. Specific ratios for the 2-, 5- and 10-year events are provided in this chapter.

Chapter 7 *Channel Routing* - General guidance is provided for the use of Normal-Depth routing, Kinematic Wave routing, Muskingum routing and Muskingum-Cunge routing. Normal-Depth routing is the preferred approach and can be applied to both natural and artificial channels. Kinematic Wave routing can be applied to urbanized or artificial channels and closed conduits. Muskingum routing can be used for large natural channels where parameter calibration data exists. Muskingum-Cunge routing may be used in all other cases.

Chapter 8 *Indirect Methods* - Three methods for verification of peak discharge estimations are provided in this chapter. The three methods incorporate local and regional data for comparison as well as generalized, regional regression equations.

Chapter 9 *Application* - General guidelines and some specific aids in the use of the manual as well as detailed examples specific to each chapter are provided.

References: A listing of all references is provided as Chapter 10.

Appendices: Isopluvial maps, loss rate tables for soils in Maricopa County, Textural Class Diagram, selected blank figures, worksheets, and other supporting information are provided in the appendices.

1.2 PURPOSE

In April 1985 a task force was formed by the Flood Control District of Maricopa County to establish a common basis for drainage management in all jurisdictions within Maricopa County. Among the goals of the task force were provisions for consistent analysis of drainage requirements, reducing costs and staff time when annexing County areas, and supplying equal and common protection from the hazards of stormwater drainage for all County residents. Additionally, developers would be benefited by having only one set of drainage standards with which to comply when developing land within the incorporated or unincorporated areas of Maricopa County. The task force determined that these efforts would be achieved in three phases:

- Phase 1 Research, evaluate, develop, and produce uniform criteria for drainage of new development which resulted in the *Uniform Drainage Policies and Standards for Maricopa County, Arizona* (herein referred to as the *Policies and Standards Manual*.)

- Phase 2 Establish a *Drainage Design Manual* for use by all jurisdictional agencies within the County.

- Phase 3 Prepare an in-depth evaluation of regional rainfall data and establish precipitation design rainfall guidelines and isohyetal maps for Maricopa County.

As a part of Phase 2, the *Drainage Design Manual for Maricopa County, Volume I, Hydrology*, will provide the necessary data for *Volume II, Hydraulics*.

1.3 SCOPE AND LIMITATION

When using the procedures detailed in this manual, it is important to keep several things in mind. First, this is a hydrologic design manual. The methods, techniques and parameter values described herein are not necessarily valid for real-time prediction of flow values, nor for recreating historic events – although some of the methods are physically based and would be amenable for uses other than design hydrology.

Second, the lack of runoff data for urbanizing areas of the County, for the most part, precludes the use of flood frequency analysis for stormwater drainage design. For those watercourses with sufficient record, flood frequency analysis may be acceptable. Similarly, for those watercourses with established regulatory floodplains, the FEMA accepted flood frequency curves may be used for design purposes, unless they are demonstrably inappropriate. The purpose of this manual is to provide a means of assisting in the prediction of runoff which might result from a design storm of a given return interval.

Third, the design storm has no point of reference in terms of a singular historic event. Rather, it is intended to provide the best available information by utilizing historic data as well as other precipitation design concepts. The design storm provides not only the peak intensities which would be expected from a storm of a given duration and return interval, but also the volumes associated with it. The tables describing the temporal distribution of the design storm for use in a hydrologic model, i.e., HEC-1, are approximately equivalent to the graphs used to determine the rainfall intensity to be used in the Rational Method. The net effect is that regardless of the size of the area being investigated or the method of analysis, the same design storm is used as the driving input.

1.4 USING THIS MANUAL

The use of the methods presented in this manual, even the rigorous application thereof, in no way ensures that the predicted values are reasonable or correct. Hydrology is a discipline which, in some respects, is much like music – quality requires not only technical competence but also a *feel* for what is right. It often requires the exercise of *hydrologic judgement*. The user of this manual is directed to validate the reasonableness of the predicted values by applying alternative methods, such as envelope curves, regression equations, or other checks which have been developed for this area and are provided in this manual. Failure to do so may result in erroneous values.

It is not the intent nor purpose of this manual to inhibit sound innovative design or the use of new techniques. Therefore, where special conditions or needs exist, other methods and procedures may be used *with prior approval*.

1.5 APPLICATION

The contents of this manual, with the exception of Chapter 3 (Rational Method) and Chapter 8 (Indirect Methods), were prepared to supplement the most current version of HEC-1 User's Manual (U.S. Army Corps of Engineers). Although the use of the HEC-1 Flood Hydrology Program is not required in conjunction with the procedures in this manual, its use will greatly facilitate the execution of the recommended procedures that are contained herein. To further enhance and simplify the use of the HEC-1 Program with the procedures in this manual, the Flood Control District has written an HEC-1 interface program, DDMSW (see Appendix E).

2 RAINFALL

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2.1 GENERAL

Precipitation in Maricopa County is strongly influenced by variation in climate, changing from a warm and semi-arid desert environment at lower elevations to a seasonally cool and moderately humid mountain environment. Mean annual precipitation ranges from about 7 inches in the Phoenix vicinity to more than 25 inches in the mountain regions of northern Maricopa County. Precipitation is typically divided into two seasons of comparative rainfall depths: summer (June through October) and winter (December through March). Warm, moist tropical air can move into Arizona at anytime of the year, but most often does so in the summer months, resulting in severe storms and local flooding. Storms of large areal extent are usually associated with frontal or convergence storm activity that may result in long duration rainfall and flooding of major drainage watercourses. These types of storms and flooding usually occur in the winter, but occasionally occur in the summer.

2.1.1 Storm and Flood Occurrence in Maricopa County

Storms in Maricopa County are often classified as general winter, general summer, and local storms. General storms are usually frontal or convergence type that cover large areas and have traditionally contributed to flooding of the major drainage watercourses in the County. Local storms are usually associated with convective activity and hence normally occur in the summer, although local storm cells (typically of lesser intensity than without frontal activity) can be imbedded in larger, general storm systems.

General winter storms usually move in from the north Pacific Ocean, and produce light to moderate precipitation over relatively large areas. These storms occur between late October and May, producing the heaviest precipitation from December to early March. Such storms could last over several days with slight breaks between individual storms. Because of orographic effects, the mountain areas generally receive more precipitation than the lower desert areas. These storms are characterized by low intensity, long duration, and large areal extent, but on occasion, with an additional surge of moisture from the southwest, can contribute to substantial runoff volumes and peak discharge on major river systems.

General summer storms are often associated with tropical storms. The Pacific Ocean north of the equator and south of Mexico is a breeding ground for such storms. On the average, about two dozen tropical storms and hurricanes are generated in this area from June through early October. Most move in a northwesterly direction. The remnants of these storms can be caught up in the large scale circulation around a low pressure center in southern California and therefore can bring a persistent flow of moist tropical air into Arizona. The storm pattern consists of a band of locally heavy rain cells within a larger area of light to moderate rainfall. Whereas general winter storms can cover much of the state, general summer storms are more localized along bands of rainfall. They are similar to winter storms in that higher elevations receive greater rainfall because of orographic influences. The period of late September through October may have storm patterns which are similar to both general summer and winter events.

Local storms consist of scattered heavy downpours of rain over areas of up to about 500 square miles for a time period of up to 6 hours. Within the storm area, exceptionally heavy rains usually cover up to 20 square miles and often last for less than 60 minutes. They are typically associated with lightning and thunder, and are referred to as thunderstorms or cloudbursts. While they can occur any time during the year, they are more frequent during summer months (July to September) when tropical moisture pushes into the area from the southeast or southwest. These storms turn into longer duration events in late summer and may be associated with general summer storms (see above). Local storms generally produce record peaks for small watersheds. They can result in flash floods, and, sometimes, loss of life and property damage.

2.1.2 Design Rainfall Criteria for Maricopa County

The critical flood-producing storm for most watersheds in Maricopa County is the local storm. The limit of such storms is generally less than 500 square miles with durations less than 6 hours. Local storms are characterized by central storm cells (possibly as large as 100 square miles) that produce very high intensity rainfalls for relatively short durations. The rainfall intensities diminish as the distance from the storm cell increases. Therefore, for the majority of watersheds and drainage areas in Maricopa County, the local storm will produce both the largest flood peak discharge and the greatest runoff volume. Based on a review of meteorologic studies for Arizona (U.S. Army Corps of Engineers, 1974 and 1982a) and a consideration of severe storms in Maricopa County, it was determined that the 6-hour local storm should be used as the design storm criteria for watersheds in Maricopa County with drainage areas of 20 square miles and less.

The 6-hour local storm for watersheds between 20 and 100 square miles may be the required design storm criteria, as discussed below. The general design storm for watershed areas between 20 and 500 square miles is the 24-hour storm.

For drainage areas between the critical flood-producing upper limit for local storms (100 square miles), and the lower limit for general storms (20 square miles), it can not be determined whether a local storm or a general storm will produce the greatest flood peak discharges or the maximum flood volumes. For such drainage areas, generally between 20 and 100 square miles, it is necessary to consider both general storms and local storms. This may require that site-specific general storm criteria be developed for the watershed and that various local storms with critical storm centering assumptions be developed using the criteria in this manual. Both of these storm types would be modeled and executed in the watershed model to estimate flood discharges and runoff volumes. It is possible, in certain situations, that the local storm could result in the largest peak discharge and the general storm could result in the largest runoff volume.

The *Policies and Standards Manual*, stipulates that the 100-year, 2-hour rainfall be used for the design of stormwater storage facilities. As such, criteria are provided in this manual to define the 100-year, 2-hour rainfall for use in Maricopa County.

Record floods for large drainage areas, such as for the Salt River near Phoenix, were produced by large-scale general storms of multiple day duration and relatively low rainfall intensities. Therefore, based on that observation, for drainage areas larger than 500 square miles it was determined that the general storm should be used as the design storm criteria. Because of the complexity of design criteria for such large areas as well as other considerations, design rainfall criteria are not defined in this manual. General storm criteria are to be defined for such large, regional flood studies on a case-by-case basis so that the most appropriate meteorologic and hydrologic factors (possibly also including snowmelt for stream baseflow and watershed antecedent moisture conditions) can be properly considered in the flood analysis.

The design rainfall criteria to be used in the unincorporated areas of Maricopa County are summarized in the *Policies and Standards Manual.* The specific procedures that are needed to define the design rainfall for the 100-year, 2-hour storm, the 6-hour local storm and the 24-hour general storm are provided in the following sections. Refer to the Policies and Standards manual of the municipality for design rainfall criteria in the incorporated areas of Maricopa County.

2.2 RAINFALL DEPTH

The most commonly used descriptor of rainfall is the rainfall depth; however, for modeling purposes, two other rainfall descriptors must be defined. First, the rainfall duration and frequency of occurrence of rainfall depth for that duration must be assigned. Second, since the rainfall depth is a descriptor of the rainfall occurrence at a point in space, both the spatial and the temporal distribution of the rainfall depth must be defined. In this section, the rainfall depth-duration-frequency statistics for use in Maricopa County are described. Subsequent sections describe the spatial and temporal distributions that are to be applied for the 6-hour local storm, the 24-hour general storm, and the temporal distribution for the 100-year, 2-hour storm.

2.2.1 Data Analyses

The most comprehensive and available source of rainfall data analysis for Maricopa County is the NOAA Precipitation-Frequency Atlas of the Western United States, (Miller and others, 1973). Until a more up-to-date data base and data analysis becomes available, the NOAA Atlas is to be used for all drainage design purposes in Maricopa County. The only deviation from the NOAA Atlas procedures that are currently recommended is the use of the short-duration (less than 1-hour) rainfall ratios that were published by Arkell and Richards (1986).

2.2.2 Depth-Duration-Frequency Statistics

The depth-duration-frequency (D-D-F) statistics in the NOAA Atlas are shown as a series of isopluvial maps of Arizona for specified durations and return periods (frequencies). Selected isopluvial maps for Maricopa County have been reproduced from the NOAA Atlas and these are contained in the *Hydrology Manual* (Figures A.1 through A.13 of Appendix A, Section 1). Areas immediately adjacent to Maricopa County are provided in the isopluvial maps, however, flood studies of certain large watersheds may require reference to the NOAA Atlas directly.

2.2.3 Rainfall Statistics for Special Purposes

There may arise situations for special purposes where it is necessary to define rainfall D-D-F statistics other than those provided in Figures A.1 through A.13. In those situations, the isopluvial maps and procedures that are contained in the NOAA Atlas along with the short-duration rainfall ratios from Arkell and Richards (1986) should be used. As an aid in the analyses and development of D-D-F statistics, a program (PREFRE) written by the Office of Hydrology, National Oceanic and Atmospheric Administration, and as modified and documented by the U.S. Bureau of Reclamation (1988), is provided. Use of the PREFRE program to calculate D-D-F statistics for special purposes is encouraged to minimize analysis errors and to increase the reproducibility of the rainfall depths that may be calculated by different users and reviewers.

The PREFRE program is incorporated into the DDMSW program. A windows based data input screen is provided to simplify the use of the PREFRE program. Input data required by the program are project location and rainfall depths for specific frequencies and durations. Rainfall depth data can be obtained from Figures A.1 through A.13 of Appendix A, Section 1. The PRE-

FRE program user manual is also included for reference in Appendix A, Section 3. Figure A.14 of Appendix A, Section 2 is a graph form for plotting rainfall-depth frequency values.

Users of this manual who may also be interested in defining general storm criteria for large watersheds, should note that it may be necessary to consider storms of durations longer than 24-hours. Provision of the 24-hour rainfall statistics does not preclude the use of a longer duration rainfall if deemed appropriate for a particular watershed or study. The 24-hour isopluvial maps are provided in this manual for the user's convenience because this is the rainfall depth often specified for general storms. If rainfall depths are needed for a duration longer than 24-hours, plot the rainfall depth versus rainfall duration for 1-hour to 24-hours (for a given rainfall frequency) on log-log paper and fit a straight line to the data points. Extend the straight line to the desired duration(s) and extrapolate the corresponding rainfall depth(s).

2.3 DEPTH-AREA RELATION

The rainfall depths from the isopluvial maps in Figures A.1 through A.13 of Appendix A, Section 1, are point rainfalls for specified frequencies and durations. This is the depth of rainfall that is expected to occur at a point or points in a watershed for the specified frequency and duration. However, this depth is not the areally-averaged rainfall over the basin that would occur during a storm. A reduction factor is used to convert the point rainfall to an equivalent uniform depth of rainfall over the entire watershed. As the watershed area increases, the reduction factor decreases, reflecting the greater nonhomogeneity of rainfall for storms of larger areas.

Regional research by the Agricultural Research Service, U.S. Department of Agriculture, for the Walnut Gulch Experimental Watershed near Tombstone, Arizona, indicated that local storms are characterized by relatively small areas of high intensity rainfall resulting in depth-area reduction curves that decrease rapidly with increasing area. The U.S. Army Corps of Engineers studied historic storms in Arizona and published the results of those studies (U.S. Army Corps of Engineers, 1974). For local storms (6-hour duration), the depth-area reduction curve that is to be used in Maricopa is the curve developed by the U.S. Army Corps of Engineers for the 19 August 1954 Queen Creek Storm. That curve is shown in Figure 2.1 and Table 2.1.

For the 24-hour general storm, the depth-area reduction curve that is to be used in Maricopa County is shown in Figure 2.2 and Table 2.2. This curve is taken from Figure 15 of the National Weather Service HYDRO-40 (Zehr and Myers, 1984).

Use the depth-area reduction values from Figure 2.1 or Table 2.1 to adjust the 6-hour, and Figure 2.2 or Table 2.2 to adjust the 24-hour, point rainfall depths from the isopluvial maps (Figures A.2 through A.13 of Appendix A, Section 1). For the design of stormwater storage facilities, refer to the *Policies and Standards Manual* for Maricopa County or the local jurisdiction for depth-area reduction values to adjust the point rainfall depth from the isopluvial map for the 100-year, 2-hour storm (Figure A.1 of Appendix A, Section 1).

For design storms other than what is specified in this manual, the depth-area reduction and temporal distribution will need to be developed on a case-by-case basis depending on the purpose of the study, location of the watershed, and other meteorological and hydrological factors.

TABLE 2.1
DEPTH-AREA REDUCTION FACTORS FOR THE 6-HOUR DURATION RAINFALL

Area, sq. miles	Ratio to Point Rainfall
0.0	1.000
0.5	**0.994**
1.0	0.987
2.8	**0.975**
5.0	0.960
10.0	0.940
16.0	**0.922**
20.0	0.910
30.0	0.890
40.0	0.870
90.0	**0.810**
100.0	**0.800**

Note: Bold values correspond to the 6-hour design storm pattern numbers.

FIGURE 2.1
DEPTH-AREA REDUCTION CURVE FOR THE 6-HOUR DURATION RAINFALL

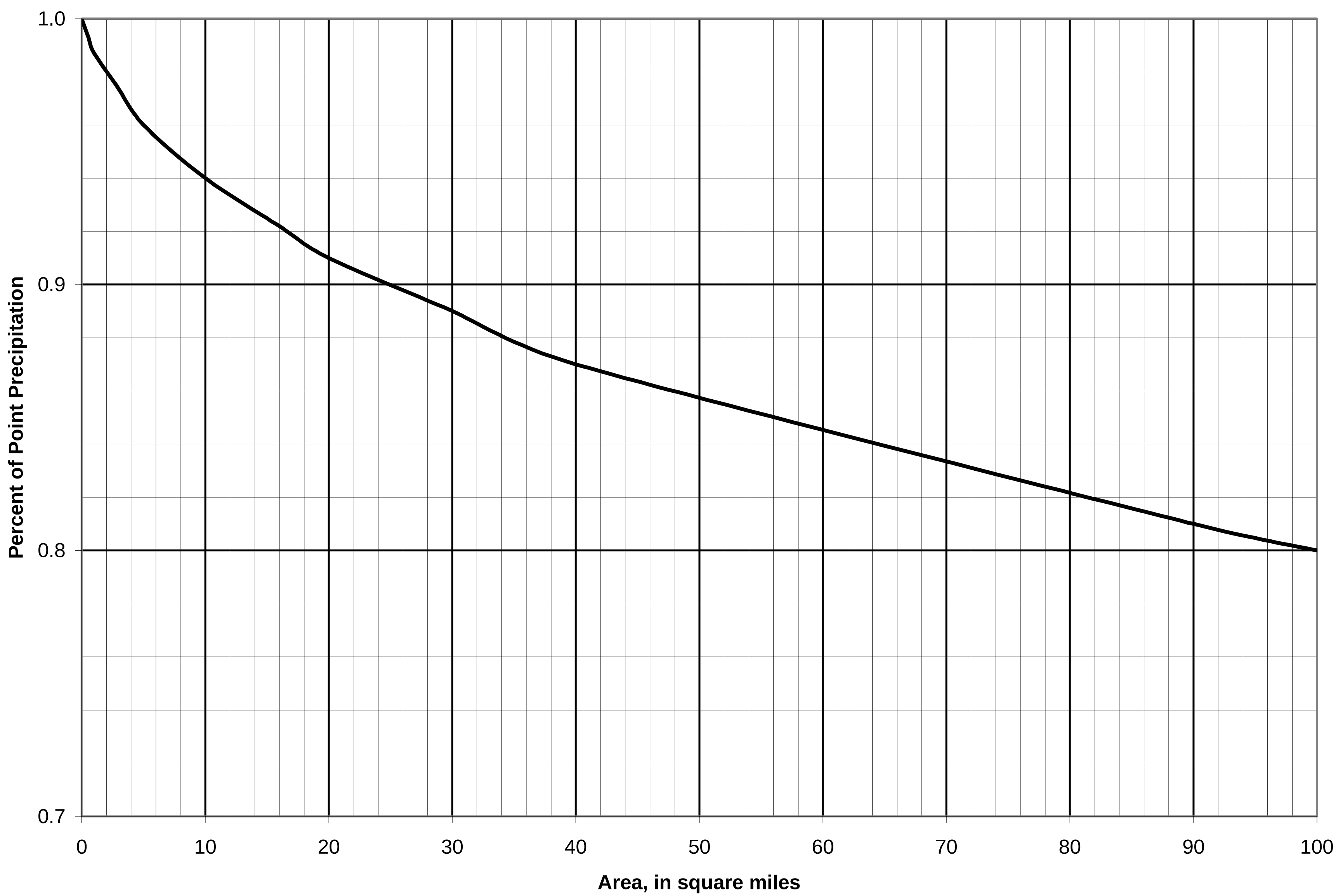
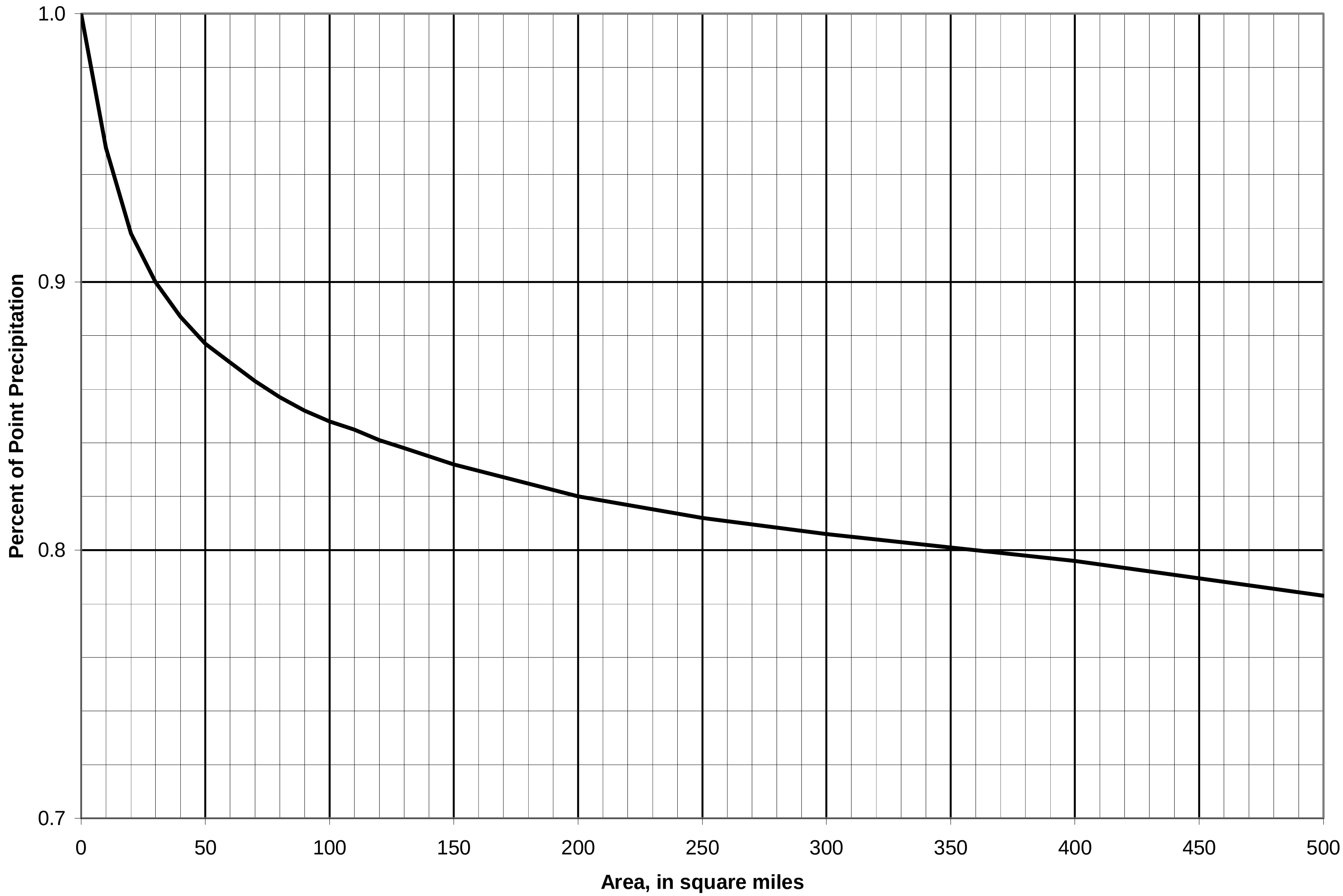


TABLE 2.2
DEPTH-AREA REDUCTION FACTORS FOR THE 24-HOUR DURATION RAINFALL

Area, sq. miles	Ratio to Point Rainfall
0	1.000
10	0.950
20	0.918
30	0.900
40	0.887
50	0.877
60	0.870
70	0.863
80	0.857
90	0.852
100	0.848
110	0.845
120	0.841
130	0.838
140	0.835
150	0.832
200	0.820
250	0.812
300	0.806
400	0.796
500	0.783

FIGURE 2.2
DEPTH-AREA REDUCTION CURVE FOR THE 24-HOUR RAINFALL DURATION



2.4 DESIGN STORM DISTRIBUTIONS

According to design rainfall criteria (*Policies and Standards Manual*), three types of design storm distributions are to be used in Maricopa County. These distributions are the 6-hour local storm, the 24-hour general storm and the 2-hour storm. Distributions for other general storms for larger watersheds will need to be developed on a case-by-case basis based on appropriate meteorologic and hydrologic factors.

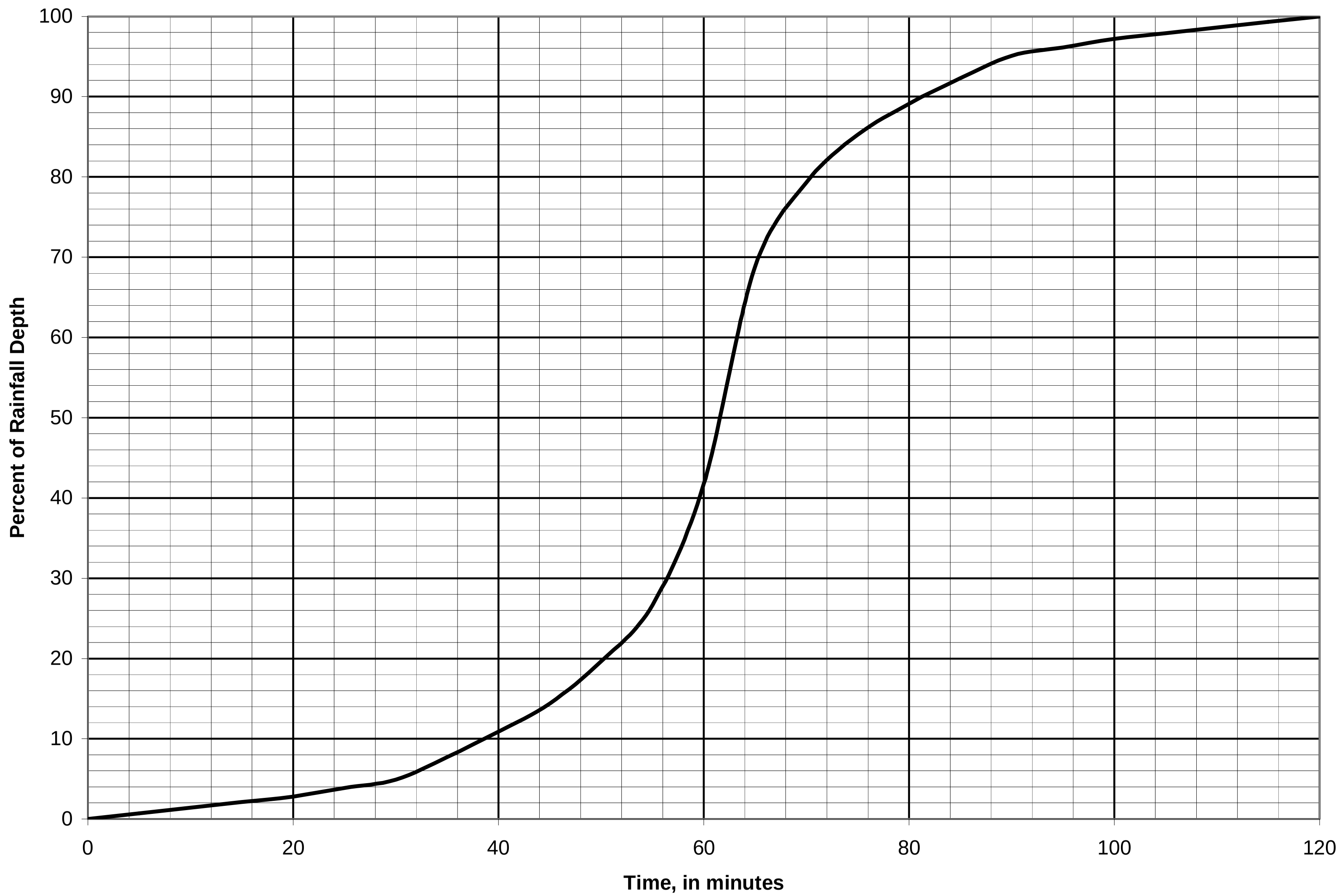
2.4.1 2-hour Storm Distribution

The 2-hour storm distribution is to be used for the design of stormwater storage facilities (see *Policies and Standards Manual*). The 2-hour distribution shown in Figure 2.3 and Table 2.3 is a dimensionless form of the 2-hour hypothetical distribution for the Phoenix Sky Harbor Airport location. This distribution can be applied throughout Maricopa County for the design of stormwater storage facilities.

TABLE 2.3
2-HOUR STORM DISTRIBUTION FOR STORMWATER STORAGE DESIGN

Time minutes	% Rainfall Depth	Time minutes	% Rainfall Depth
0	0.0	65	68.8
5	0.7	70	79.3
10	1.4	75	85.3
15	2.1	80	89.1
20	2.8	85	92.3
25	3.9	90	95.1
30	4.9	95	96.1
35	7.7	100	97.2
40	10.9	105	97.9
45	14.4	110	98.6
50	19.6	115	99.3
55	26.7	120	100.0
60	41.8		

FIGURE 2.3
2-HOUR MASS CURVE FOR STORMWATER STORAGE DESIGN



2.4.2 6-hour Storm Distribution

The 6-hour storm distributions are used for flood studies in Maricopa County of drainage areas less than 20 square miles, except for on-site stormwater storage facilities (see *Policies and Standards Manual*). These distributions would also be used for drainage areas larger than 20 square miles and smaller than 100 square miles by critically centering the storm over all or portions of the drainage area to estimate the peak flood discharges that could be realized on such watersheds due to the occurrence of a local storm over the watershed.

The Maricopa County 6-hour local storm distributions consist of five dimensionless storm patterns. Pattern No. 1 represents the rainfall intensities that can be expected in the “eye” of a local storm. These high, short-duration rainfall intensities would only occur over a relatively small area near the center of the storm cell. Pattern No. 1 is an offset, dimensionless form of the hypothetical distribution derived from rainfall statistics found in the NOAA Atlas for the Western United States, Arizona (Miller and others, 1973) and Arkell and Richards (1986) for the Phoenix Sky Harbor Airport location. Pattern Numbers 2 through 5 are modifications of the U.S. Army Corps of Engineers (1974) analysis of the Queen Creek storm of 19 August 1954. The dimensionless form of these 6-hour storm distributions are shown in Figure 2.4 and Table 2.4.

Inspection of the storm patterns in Figure 2.4 indicates that the peak rainfall intensities are much greater for Pattern No. 1 than for the other pattern numbers, and that peak rainfall intensity decreases as the pattern number increases. The selection of the pattern number is based on the size of the drainage area under consideration, as shown in Figure 2.5. As illustrated by Figures 2.4 and 2.5, the maximum rainfall intensities, averaged over the entire drainage area, decrease as the size of the drainage area increases. This is to account for the spatial variability of local storm rainfall wherein the maximum rainfall intensities occur at the relatively small eye of the storm but that the average rainfall intensities over the storm area decrease as the storm area increases.

TABLE 2.4
6-HOUR DISTRIBUTIONS *

	Percent of Rainfall Depth				
Time, in hours	**Pattern 1**	**Pattern 2**	**Pattern 3**	**Pattern 4**	**Pattern 5**
0:00	0.0	0.0	0.0	0.0	0.0
0:15	0.8	0.9	1.5	2.1	2.4
0:30	1.6	1.6	2.0	3.5	4.3
0:45	2.5	2.5	3.0	5.1	5.9
1:00	3.3	3.4	4.8	7.1	7.8
1:15	4.1	4.2	6.3	8.7	9.8
1:30	5.0	5.1	7.6	10.5	11.9
1:45	5.8	5.9	9.0	12.5	14.1

2:00	6.6	6.7	10.5	14.3	16.2
2:15	7.4	7.6	11.9	16.0	18.6
2:30	8.7	8.7	13.5	17.9	21.2
2:45	9.9	10.0	15.2	20.1	23.9
3:00	11.8	12.0	17.5	23.2	27.1
3:15	13.8	16.3	22.2	28.1	32.1
3:30	21.6	25.2	30.4	36.4	40.8
3:45	37.7	45.1	47.2	50.0	51.5
4:00	83.4	69.4	67.0	65.8	62.7
4:15	91.1	83.7	79.6	77.3	73.5
4:30	93.1	90.0	86.8	84.1	81.4
4:45	95.0	93.8	91.2	88.8	86.4
5:00	96.2	95.0	94.6	92.7	90.7
5:15	97.2	96.3	96.0	94.5	93.0
5:30	98.3	97.5	97.3	96.4	95.4
5:45	99.1	98.8	98.7	98.2	97.7
6:00	100.0	100.0	100.0	100.0	100.0

*Pattern represents percent Rainfall Depth.

FIGURE 2.4
6-HOUR MASS CURVES FOR MARICOPA COUNTY

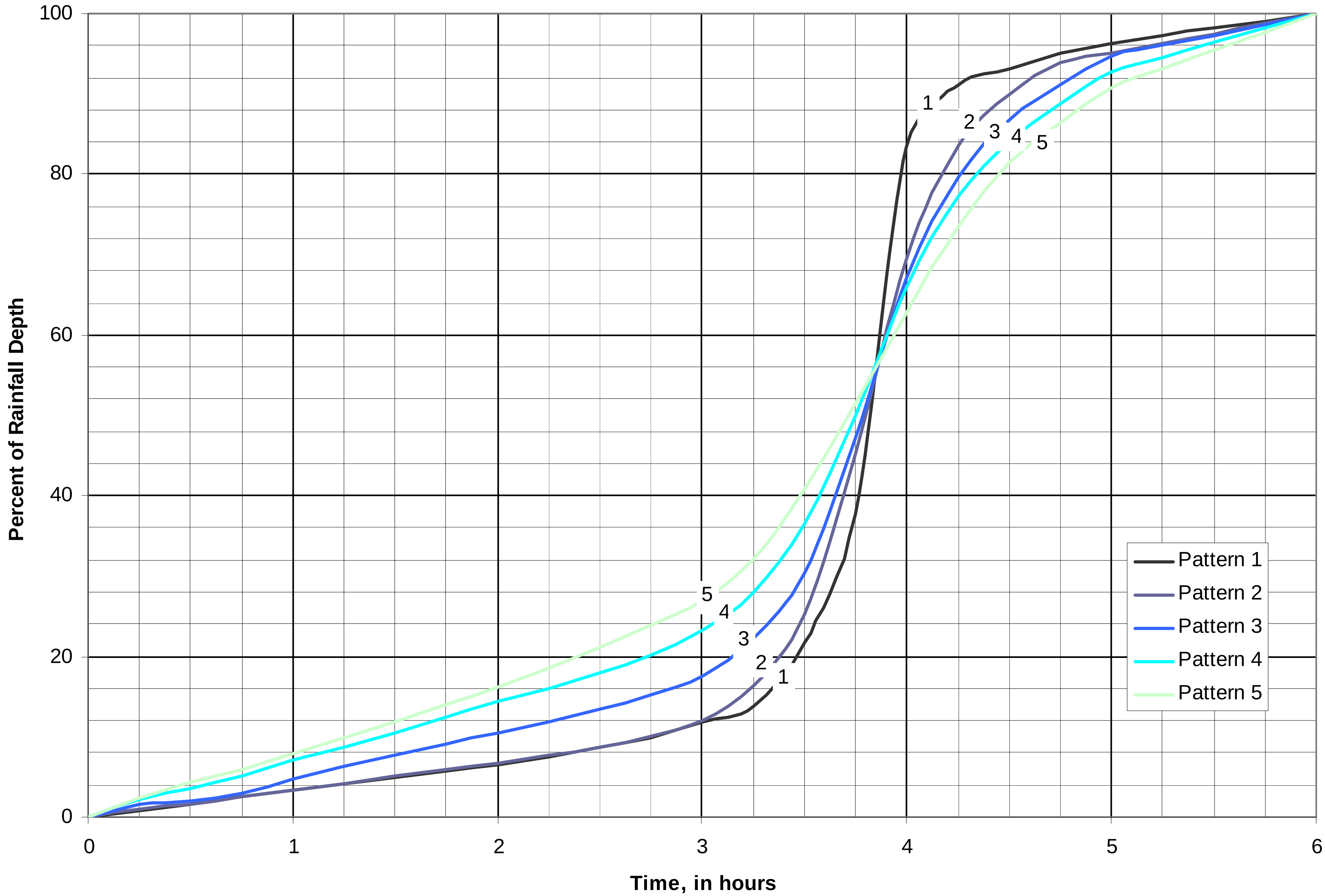
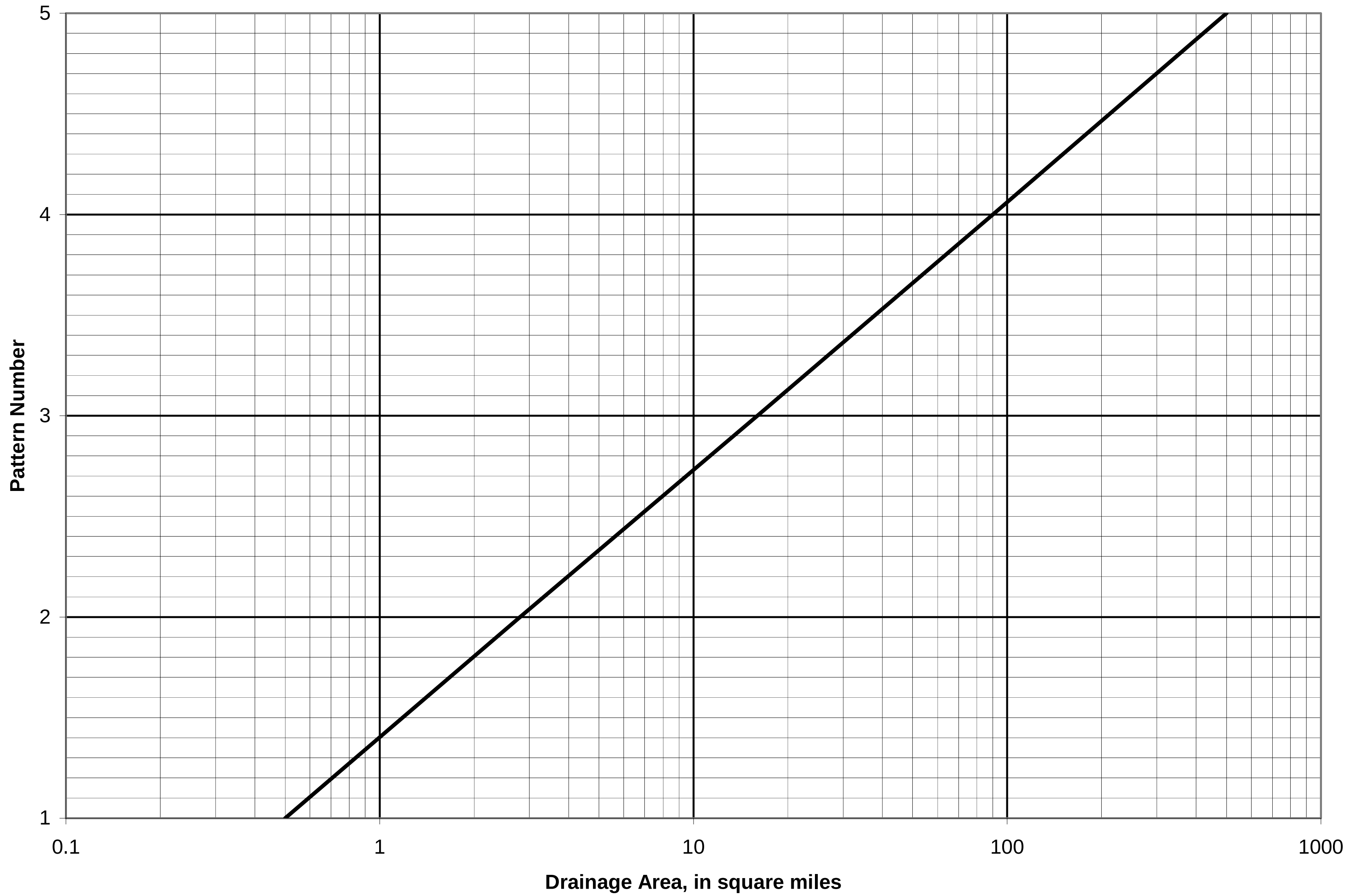


FIGURE 2.5
AREA VERSUS PATTERN NUMBER FOR MARICOPA COUNTY



2.4.3 24-hour Storm Distribution

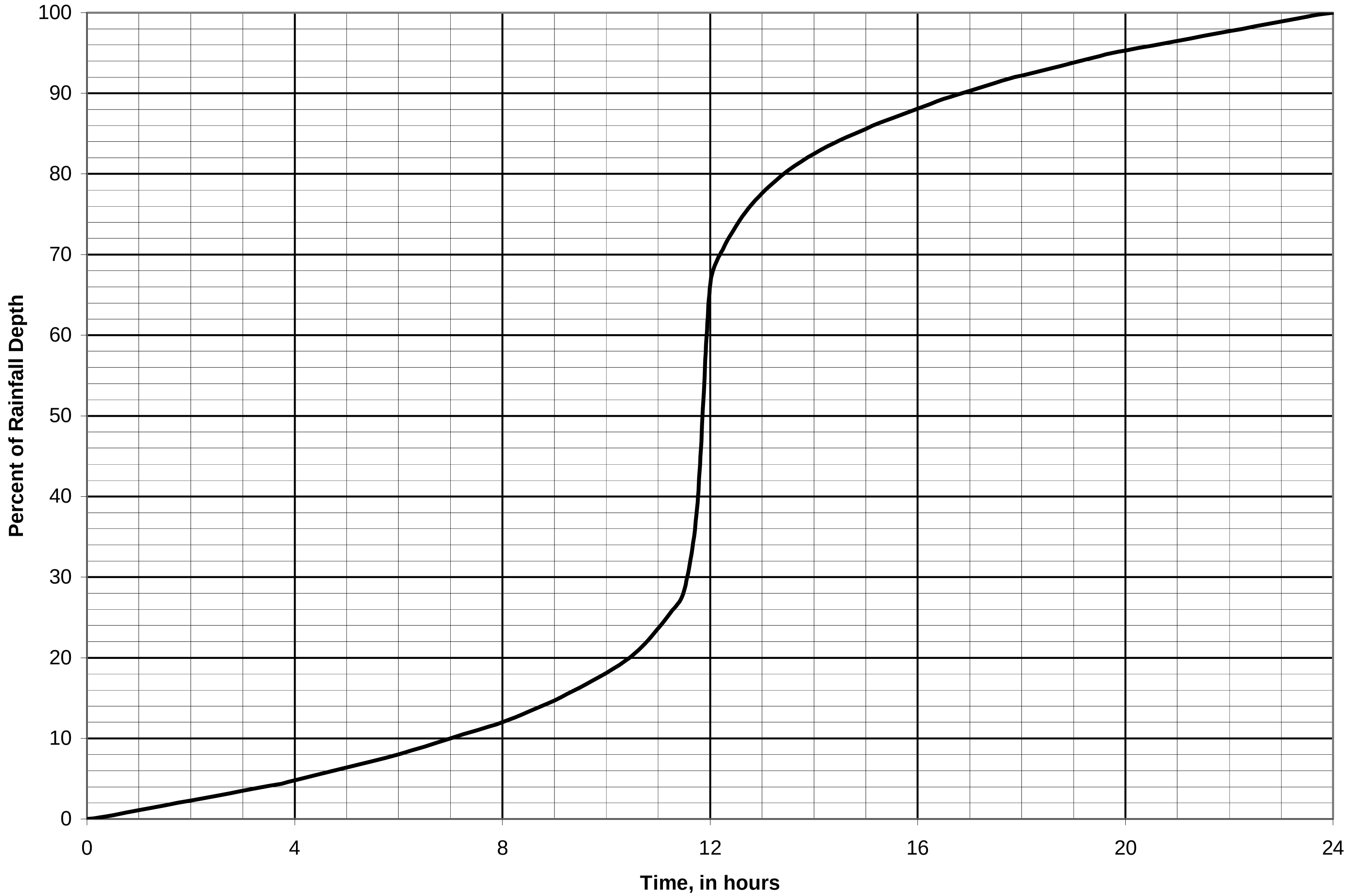
The 24-hour storm distribution that is to be used in Maricopa County is the SCS Type II distribution. This distribution is shown in Table 2.5 and Figure 2.6. The 24-hour storm distribution is used for flood studies of drainage area larger than 100 square miles (see *Policies and Standards Manual*). This distribution is also to be used in combination with the 6-hour storm distribution for drainage areas between 20 and 100 square miles to determine whether a local storm or a general storm will produce the greatest flood peak discharges or the maximum flood volumes.

TABLE 2.5
24-HOUR DISTRIBUTION

Time hours	Rainfall Depth %	Time hours	Rainfall Depth %	Time hours	Rainfall Depth %
0.00	0.0	8.25	12.6	16.50	89.3
0.25	0.2	8.50	13.3	16.75	89.8
0.50	0.5	8.75	14.0	17.00	90.3
0.75	0.8	9.00	14.7	17.25	90.8
1.00	1.1	9.25	15.5	17.50	91.3
1.25	1.4	9.50	16.3	17.75	91.8
1.50	1.7	9.75	17.2	18.00	92.2
1.75	2.0	10.00	18.1	18.25	92.6
2.00	2.3	10.25	19.1	18.50	93.0
2.25	2.6	10.50	20.3	18.75	93.4
2.50	2.9	10.75	21.8	19.00	93.8
2.75	3.2	11.00	23.6	19.25	94.2
3.00	3.5	11.25	25.7	19.50	94.6
3.25	3.8	11.50	28.3	19.75	95.0
3.50	4.1	11.75	38.7	20.00	95.3
3.75	4.4	12.00	66.3	20.25	95.6
4.00	4.8	12.25	70.7	20.50	95.9
4.25	5.2	12.50	73.5	20.75	96.2
4.50	5.6	12.75	75.8	21.00	96.5
4.75	6.0	13.00	77.6	21.25	96.8
5.00	6.4	13.25	79.1	21.50	97.1
5.25	6.8	13.50	80.4	21.75	97.4
5.50	7.2	13.75	81.5	22.00	97.7
5.75	7.6	14.00	82.5	22.25	98.0
6.00	8.0	14.25	83.4	22.50	98.3
6.25	8.5	14.50	84.2	22.75	98.6

6.50	9.0	14.75	84.9	23.00	98.9
6.75	9.5	15.00	85.6	23.25	99.2
7.00	10.0	15.25	86.3	23.50	99.5
7.25	10.5	15.50	86.9	23.75	99.8
7.50	11.0	15.75	87.5	24.00	100.0
7.75	11.5	16.00	88.1		
8.00	12.0	16.25	88.7		

FIGURE 2.6
24-HOUR MASS CURVE FOR MARICOPA COUNTY (SCS TYPE II)



2.5 PROCEDURE FOR THE DEVELOPMENT OF THE DESIGN RAINFALL

The following is the procedure for the development of the design rainfall. Notes and general guidance on the application of this procedure and the methodologies presented in this chapter are provided along with a detailed example in Chapter 9, Section 9.1.

1. Determine the size of the drainage area.

2. Determine the point rainfall depth or the areally averaged point rainfall depth, from Figures A.2 through A.13 of Appendix A, Section 1, depending on the desired storm duration and frequency.

3. For a single storm analysis, determine the depth-area reduction factor using Figure 2.1 or Table 2.1 for a 6-hour local storm and Figure 2.2 or Table 2.2 for a 24-hour general storm.

   For a multiple storm analysis, determine the drainage areas at key points of interest in the watershed. For each drainage area, determine the depth-area reduction factor using Figure 2.1 or Table 2.1 for a 6-hour local storm and Figure 2.2 or Table 2.2 for a 24-hour general storm.

   As drainage area increases, the average depth of rainfall over that area decreases. For situations that require runoff magnitudes at only one point in the watershed, the effective rainfall over the watershed can be simulated by a single storm. The single storm approach can be applied regardless of the number of subbasins used to define the runoff characteristics of the watershed.

   For situations that require runoff magnitudes at multiple points within a drainage area, the effective rainfall depth at each of those points is simulated using a set of index storms. The drainage areas of the index storms and thus the rainfall depth adjustment factors are selected to be representative of the contributing drainage areas at the points of interest. This implies that the watershed will be delineated with multiple subbasins.

4. Multiply the point rainfall depth by the appropriate depth-area reduction factor(s).

5. For a 6-hour local storm, use Figure 2.5 to select the appropriate pattern number(s) (rounded to the nearest 0.1 pattern number).

6. For a 6-hour local storm, use the dimensionless rainfall distributions of Figure 2.4 or Table 2.4 to calculate the dimensionless distribution(s) by linear interpolation between the two bounding pattern numbers.

For a 24-hour general storm, use the dimensionless rainfall distribution of Figure 2.6 or Table 2.5.

Note: Steps 3 through 6 are performed automatically in DDMSW.

3 RATIONAL METHOD

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3.1 GENERAL

The Rational Method was originally developed to estimate runoff from small areas and its use should be generally limited to those conditions. For the purposes of this manual, its use should be limited to areas of up to 160 acres. In such cases, the peak discharge and the volume of runoff from rainfall events up to and including the 100-year, 2-hour duration storm falling within the boundaries of the proposed development are to be retained. This is the required criteria for unincorporated areas of Maricopa County. For incorporated areas, the 100-year, 2-hour duration storm is the minimum recommended criteria, however the Policies and Standards manual for the jurisdictional entity should be referenced for any variations. If the development involves channel routing, the procedures given in Chapters 4 through 6 should be used, since the peak generated by the Rational Method cannot be directly routed.

3.2 RATIONAL EQUATION

The Rational Equation relates rainfall intensity, a runoff coefficient and the watershed size to the generated peak discharge. The following shows this relationship:

$$Q = CiA \tag{3.1}$$

where:

Q = the peak discharge, in cfs, from a given area.

C = a coefficient relating the runoff to rainfall.

i = average rainfall intensity, in inches/hour, lasting for a T_c.

T_c = the time of concentration, in hours.

A = drainage area, in acres.

The Rational Equation is based on the concept that the application of a steady, uniform rainfall intensity will produce a peak discharge at such a time when all points of the watershed are contributing to the outflow at the point of design. Such a condition is met when the elapsed time is equal to the time of concentration, T_c, which is defined to be the floodwave travel time from the most remote part of the watershed to the point of design. The time of concentration should be computed by applying the following equation developed by Papadakis and Kazan (1987):

$$T_c = 11.4\, L^{0.5}\, K_b^{0.52}\, S^{-0.31}\, i^{-0.38} \qquad (3.2)$$

where:

T_c = time of concentration, in hours.

L = length of the longest flow path, in miles.

K_b = watershed resistance coefficient (see Figure 3.1, or Table 3.1).

S = watercourse slope, in feet/mile.

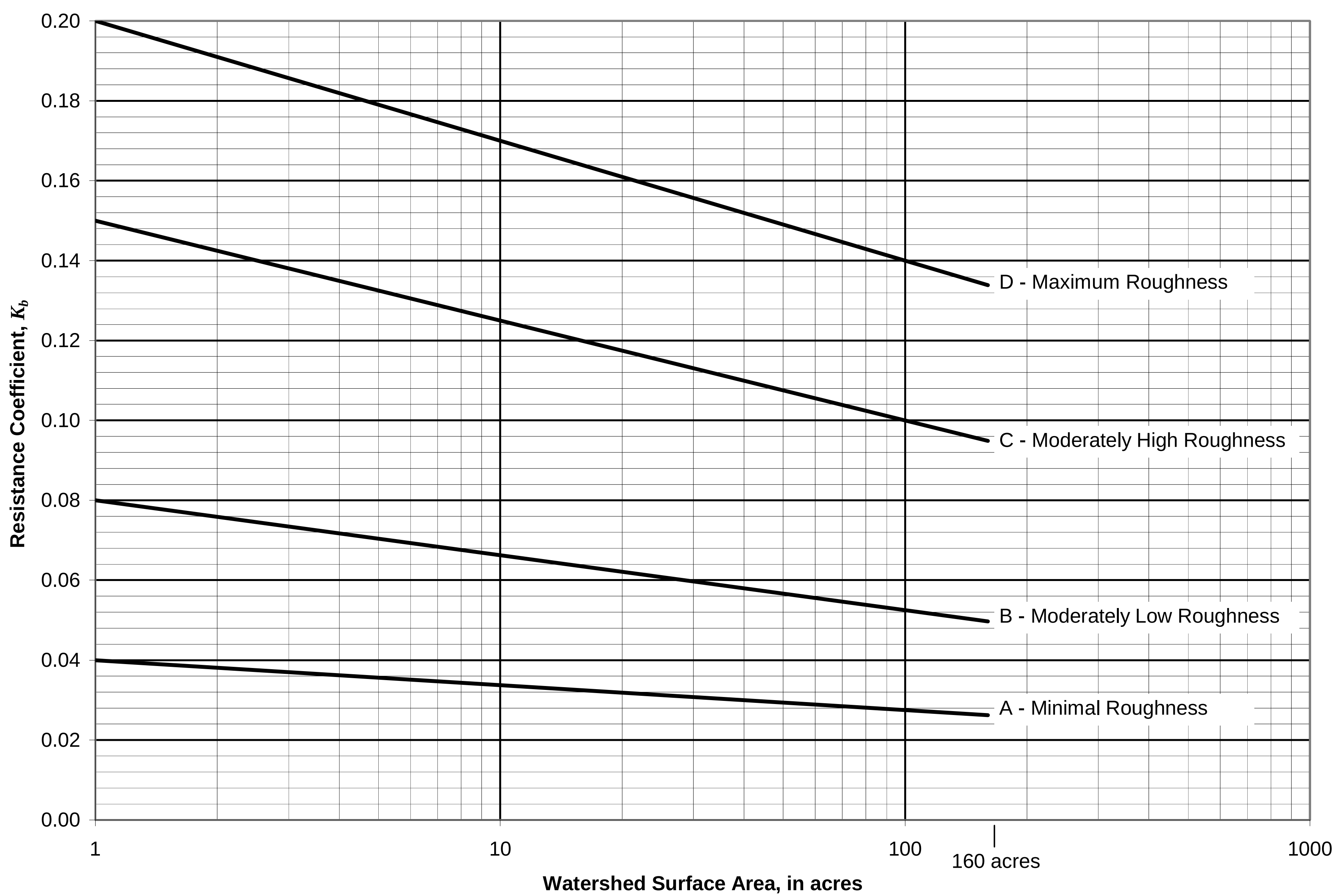
i = rainfall intensity, in inches/hour.*

*It should be noted that i is the "rainfall excess intensity" as originally developed. However, when used in the Rational Equation, rainfall intensity and rainfall excess intensity provide similar values because of the hydrologic characteristics of small, urban watersheds which result in minimal rainfall loss. This is because of the extent of imperviousness associated with urban watersheds and the fact that the time of concentration is usually very short.

TABLE 3.1
EQUATION FOR ESTIMATING K_B IN THE T_C EQUATION

$K_b = m \log A + b$ **Where A is drainage area, in acres**				
Type	**Description**	**Typical Applications**	**Equation Parameters** m	**Equation Parameters** b
A	Minimal roughness: Relatively smooth and/or well graded and uniform land surfaces. Surfaces runoff is sheet flow.	Commercial/industrial areas Residential area Parks and golf courses	-0.00625	0.04
B	Moderately low roughness: Land surfaces have irregularly spaced roughness elements that protrude from the surface but the overall character of the surface is relatively uniform. Surface runoff is predominately sheet flow around the roughness elements.	Agricultural fields Pastures Desert rangelands Undeveloped urban lands	-0.01375	0.08
C	Moderately high roughness: Land surfaces that have significant large to medium-sized roughness elements and/or poorly graded land surfaces that cause the flow to be diverted around the roughness elements. Surface runoff is sheet flow for short distances draining into meandering drainage paths.	Hillslopes Brushy alluvial fans Hilly rangeland Disturbed land, mining, etc. Forests with underbrush	-0.025	0.15
D	Maximum roughness: Rough land surfaces with torturous flow paths. Surface runoff is concentrated in numerous short flow paths that are often oblique to the main flow direction.	Mountains Some wetlands	-0.030	0.20

FIGURE 3.1
RESISTANCE COEFFICIENT K_b AS A FUNCTION OF WATERSHED SIZE AND SURFACE ROUGHNESS CHARACTERISTICS



Rational Method runoff coefficients for various natural conditions and land uses are provided in Table 3.2.

TABLE 3.2
RUNOFF COEFFICIENTS FOR MARICOPA COUNTY

Land Use Code	Land Use Category	Runoff Coefficients by Storm Frequency[1, 2]							
		2-10 Year		25 Year		50 Year		100 Year	
		min	max	min	max	min	max	min	max
VLDR	Very Low Density Residential[3]	0.33	0.42	0.36	0.46	0.40	0.50	0.41	0.53
LDR	Low Density Residential[3]	0.42	0.48	0.46	0.53	0.50	0.58	0.53	0.60
MDR	Medium Density Residential[3]	0.48	0.65	0.53	0.72	0.58	0.78	0.60	0.82
MFR	Multiple Family Residential[3]	0.65	0.75	0.72	0.83	0.78	0.90	0.82	0.94
I1	Industrial 1[3]	0.60	0.70	0.66	0.77	0.72	0.84	0.75	0.88
I2	Industrial 2[3]	0.70	0.80	0.77	0.88	0.84	0.95	0.88	0.95
C1	Commercial 1[3]	0.55	0.65	0.61	0.72	0.66	0.78	0.69	0.81
C2	Commercial 2[3]	0.75	0.85	0.83	0.94	0.90	0.95	0.94	0.95
P	Pavement and Rooftops	0.75	0.85	0.83	0.94	0.90	0.95	0.94	0.95
GR	Gravel Roadways & Shoulders	0.60	0.70	0.66	0.77	0.72	0.84	0.75	0.88
AG	Agricultural	0.10	0.20	0.11	0.22	0.12	0.24	0.13	0.25
LPC	Lawns/Parks/Cemeteries	0.10	0.25	0.11	0.28	0.12	0.30	0.13	0.31
DL1	Desert Landscaping 1	0.55	0.85	0.61	0.94	0.66	0.95	0.69	0.95
DL2	Desert Landscaping 2	0.30	0.40	0.33	0.44	0.36	0.48	0.38	0.50
NDR	Undeveloped Desert Rangeland	0.30	0.40	0.33	0.44	0.36	0.48	0.38	0.50
NHS	Hillslopes, Sonoran Desert	0.40	0.55	0.44	0.61	0.48	0.66	0.50	0.69
NMT	Mountain Terrain	0.60	0.80	0.66	0.88	0.72	0.95	0.75	0.95

Notes:

1. Runoff coefficients for 25-, 50- and 100-Year storm frequencies were derived using adjustment factors of 1.10, 1.20 and 1.25, respectively, applied to the 2-10 Year values with an upper limit of 0.95.
2. The ranges of runoff coefficients shown for urban land uses were derived from lot coverage standards specified in the zoning ordinances for Maricopa County.
3. Runoff coefficients for urban land uses are for lot coverage only and do not include the adjacent street and right-of-way, or alleys.

TABLE 3.3
RUNOFF COEFFICIENT DESCRIPTIONS FOR MARICOPA COUNTY

Land Use Code	Land Use Category Description
VLDR	40,000 sq. feet and greater lot size
LDR	12,000 – 40,000 sq. feet lot size
MDR	6,000 – 12,000 sq. feet lot size
MFR	1,000 – 6,000 sq. feet lot size
I1	Light and General
I2	General and Heavy
C1	Light, Neighborhood, Residential
C2	Central, General, Office, Intermediate
P	Asphalt and Concrete, Sloped Rooftops
GR	Graded and Compacted, Treated and Untreated
AG	Tilled Fields, Irrigated Pastures, slopes < 1%
LPC	Over 80% maintained lawn
DL1	Landscaping with impervious under treatment
DL2	Landscaping without impervious under treatment
NDR	Little topographic relief, slopes < 5%
NHS	Moderate topographic relief, slopes > 5%
NMT	High topographic relief, slopes > 10%

3.3 ASSUMPTIONS

Application of the Rational Equation requires consideration of the following:

1. The peak discharge rate corresponding to a given intensity would occur only if the rainfall duration is at least equal to the time of concentration.

2. The calculated runoff is directly proportional to the rainfall intensity.

3. The frequency of occurrence for the peak discharge is the same as the frequency for the rainfall producing that event.

4. The runoff coefficient increases as storm frequency decreases.

3.4 VOLUME CALCULATIONS

Volume calculations should be done by applying the following equation:

$$V = C\left(\frac{P}{12}\right)A \tag{3.3}$$

where:

V = calculated volume, in acre-feet.

C = runoff coefficient from Table 3.2.

P = rainfall depth, in inches.

A = drainage area, in acres.

In the case of volume calculations for stormwater storage facility design, P equals the 100-year, 2-hour depth, in inches, as discussed in Section 2.2, and is determined from Figure A.2 of Appendix A, Section1.

3.5 LIMITATIONS

Application of the Rational Method is appropriate for watersheds less than 160 acres in size. This is based on the assumption that the rainfall intensity is to be uniformly distributed over the drainage area at a uniform rate lasting for the duration of the storm. The Maricopa County Unit Hydrograph Procedure described in Chapter 5 may also be used for areas less then 160 acres where hydrograph routing is desired, or, in cases where the Rational Method assumptions do not apply.

3.6 APPLICATION

The Rational Method can be used to calculate the generated peak discharge from drainage areas less than 160 acres. Procedures for calculating peak discharge are provided in the following sections. Notes and general guidance in the application of these procedures along with a detailed example are provided in Chapter 9, Section 9.2.

3.6.1 Peak Discharge Calculation

1. Determine the area within the development boundaries.

2. Select the Runoff Coefficient, C from Table 3.2. If the drainage area contains subareas of different runoff characteristics, and thus different C coefficients, arithmetically area-weight the values of C.

3. Compute the depth-duration-frequency (D-D-F) statistics for the project site using the PREFRE program (see Chapter 2, Section 2.2). Alternatively, if the project site lies within the Phoenix Metro area, then the I-D-F graph in Appendix B can be used to compute intensity.

4. Calculate the time of concentration. This is to be done as an iterative process.

   a. Determine the K_b parameter from Figure 3.1 or Table 3.1. If the drainage area contains subareas of different K_b values, arithmetically area-weight the values of K_b.

   b. Make an initial estimate of the duration and compute the intensity from the PREFRE output for the desired frequency. If the project site is within the Phoenix metro area, the I-D-F graph provided in Appendix B can be used as an alternative.

   c. Compute an estimated T_c using Equation 3.2. If the computed T_c is reasonably close to the estimated duration, then proceed to Step 5, otherwise repeat this step with a new estimate of the duration. The minimum T_c should not be less than 10-minutes.

5. Determine peak discharge Q by using the above value of i in Equation 3.1.

6. As an alternative to the above procedure, the DDMSW program may be used to calculate peak discharges.

3.6.2 Multiple Basin Approach

The Rational Method can be used to compute peak discharges at intermediate locations within a drainage area less than 160 acres in size. A typical application of this approach is a local storm drain system where multiple subbasins are necessary to compute a peak discharge at each proposed inlet location. Consider the schematic example watershed shown in Figure 3.2. A peak discharge is needed for all three individual subareas, subareas A and B combined at Concentration Point 1 and subareas A, B and C combined at Concentration Point 2.

1. Compute the peak discharge for each individual subarea using steps 1 through 5 from Section 3.6.1.

2. Compute the arithmetically area-weighted value of C for subareas A and B.

3. Follow step 4 from Section 3.6.1 to calculate the T_c for the combined area of subareas A and B at Concentration Point 1.

4. Compare the T_c values from subareas A and B to the T_c value for the combined area at Concentration Point 1. Compute the peak discharge at Concentration Point 1 using the i for the longest T_c from step 3. If the combined peak discharge is less than the discharges for the individual subareas, use the largest discharge as the peak discharge at Concentration Point 1. The design discharge SHOULD NOT INCREASE going downstream in a conveyance system unless storage facilities are used to attenuate peak flows.

5. Compute the arithmetically area-weighted value of C for subareas A, B and C.

6. Calculate the T_c for the combined area at Concentration Point 2 using the following two methods:

   Method 1 - Follow step 4 from Section 3.6.1 to calculate the T_c for the single basin composed of all three subareas.

   Method 2 - Compute the travel time from Concentration Point 1 to Concentration Point 2 using the continuity equation or other appropriate technique and hydraulic parameters for the conveyance path. Add the computed travel time for the conveyance path to the T_c from Concentration Point 1.

7. Compare the T_c values from Methods 1 and 2 as well as the T_c from subarea C and calculate the peak discharge at Concentration Point 2 as follows:

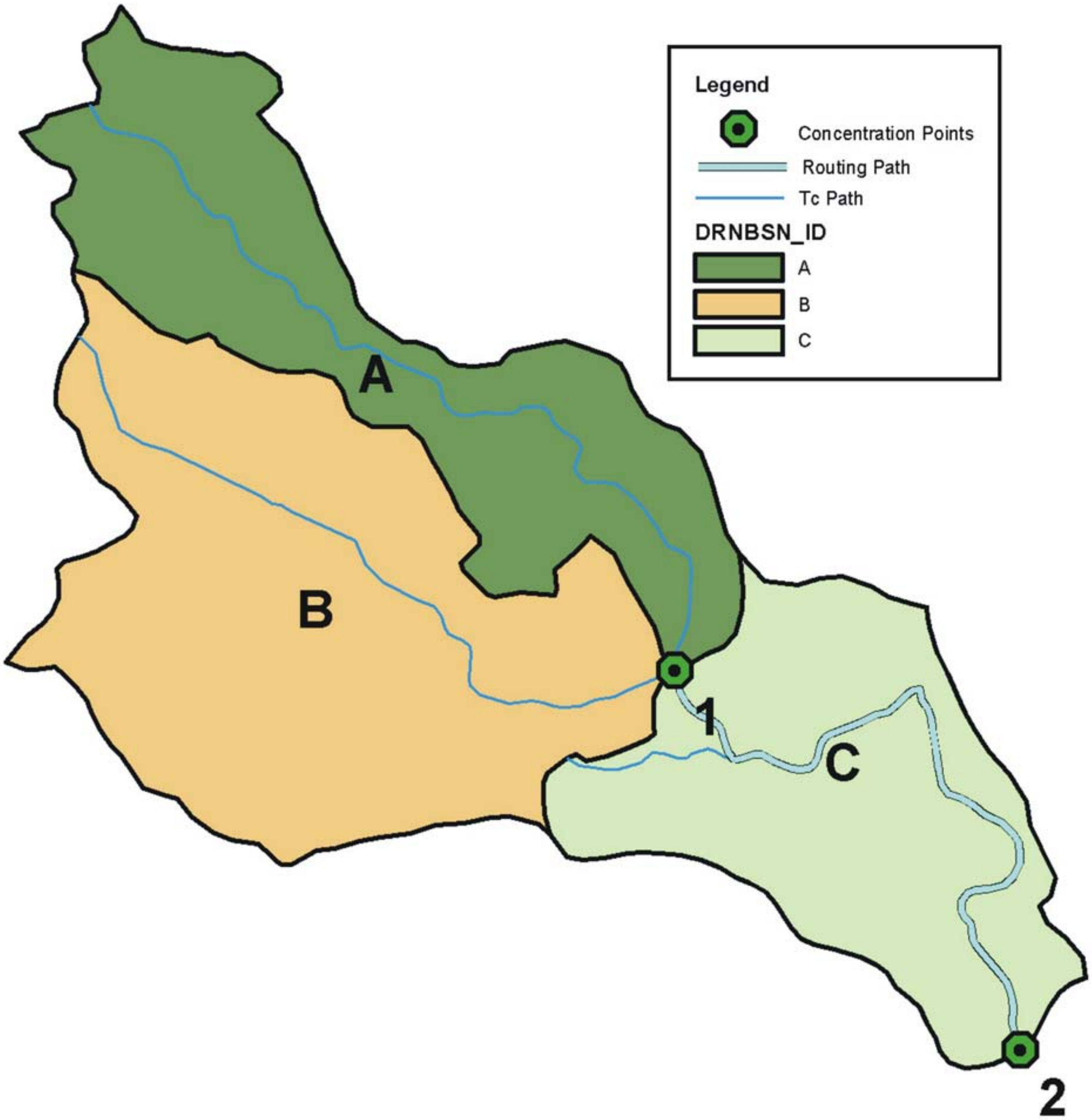
   a. If the T_c value from Method 1 is the longest, compute the total peak discharge using the Method 1 intensity, the arithmetically area-weighted value of C for all three subareas and the total contributing drainage area at Concentration Point 2.

   b. If the T_c value from Method 2 is the longest, determine i directly from the D-D-F statistics from step 3 of Section 3.6.1. Compute the total peak discharge at Concentration Point 2 using the arithmetically area-weighted value of C for all three subareas and the total contributing drainage area at Concentration Point 2.

   c. If the T_c from subarea C is the longest, compute the total peak discharge using the i for subarea C, the arithmetically area-weighted value of C for all three subareas and the total contributing drainage area at Concentration Point 2.

8. As an alternative to the above procedure, the DDMSW program may be used to calculate the peak discharge at intermediate locations.

FIGURE 3.2
SCHEMATIC EXAMPLE WATERSHED



4 RAINFALL LOSSES

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4.1 GENERAL

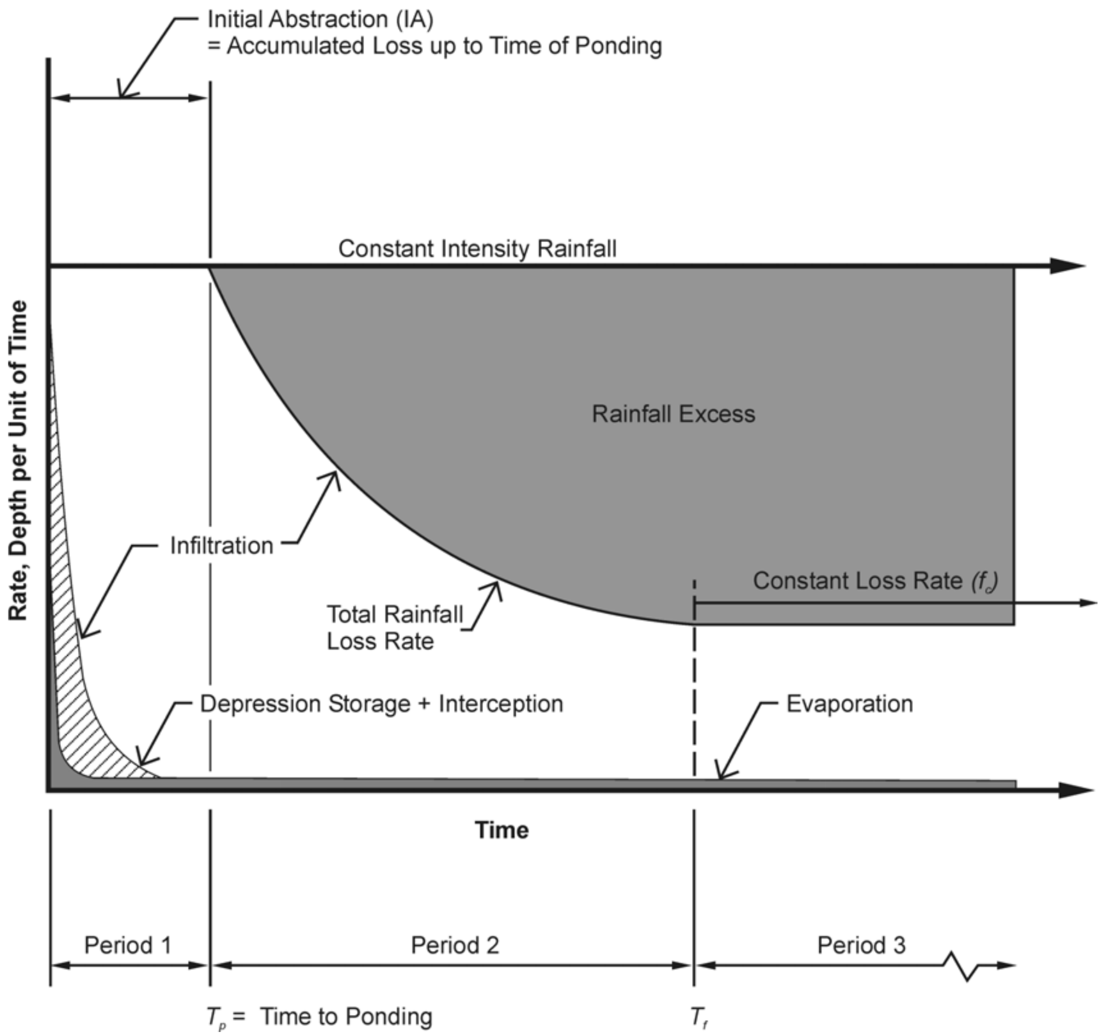
Rainfall excess is that portion of the total rainfall depth that drains directly from the land surface by overland flow. By a mass balance, rainfall excess plus rainfall loss equals precipitation. When performing a flood analysis using a rainfall-runoff model, the determination of rainfall excess is of utmost importance. Rainfall excess integrated over the entire watershed results in runoff volume, and the temporal distribution of the rainfall excess will, along with the hydraulics of runoff, determine the peak discharge. Therefore, the estimation of the magnitude and time distribution of rainfall losses should be performed with the best practical technology, considering the objective of the analysis, economics of the project, and consequences of inaccurate estimates.

Rainfall losses are generally considered to be the result of evaporation of water from the land surface, interception of rainfall by vegetal cover, depression storage on the land surface (paved or unpaved), and the infiltration of water into the soil matrix. A schematic representation of rainfall losses for a uniform intensity rainfall is shown in Figure 4.1. As shown in the figure, evaporation can start at an initially high rate depending on the land surface temperature, but the rate decreases very rapidly and would eventually reach a low, steady-state rate. From a practical standpoint, the magnitude of rainfall loss that can be realized from evaporation during a storm of sufficient magnitude to cause flood runoff is negligible.

Interception, also illustrated in Figure 4.1, varies depending upon the type of vegetation, maturity, and extent of canopy cover. Experimental data on interception have been collected by numerous investigators (Linsley and others, 1982), but little is known of the interception values for most hydrologic problems. Estimates of interception for various vegetation types (Linsley and others, 1982) are:

Vegetation Type	Interception, inches
Hardwood tree	0.09
Cotton	0.33
Alfalfa	0.11
Meadow grass	0.08

FIGURE 4.1
SCHEMATIC REPRESENTATION OF RAINFALL LOSSES FOR A UNIFORM INTENSITY RAINFALL



No interception estimates are known for natural vegetation that occurs in Maricopa County. For most applications in Maricopa County the magnitude of interception losses is essentially 0.0. Interception is considered for flood hydrology in Maricopa County, but for practical purposes an actual value is not assigned.

Depression storage and infiltration losses comprise the majority of the rainfall loss as illustrated in Figure 4.1. The estimates of these two losses will be discussed in more detail in later sections of this manual.

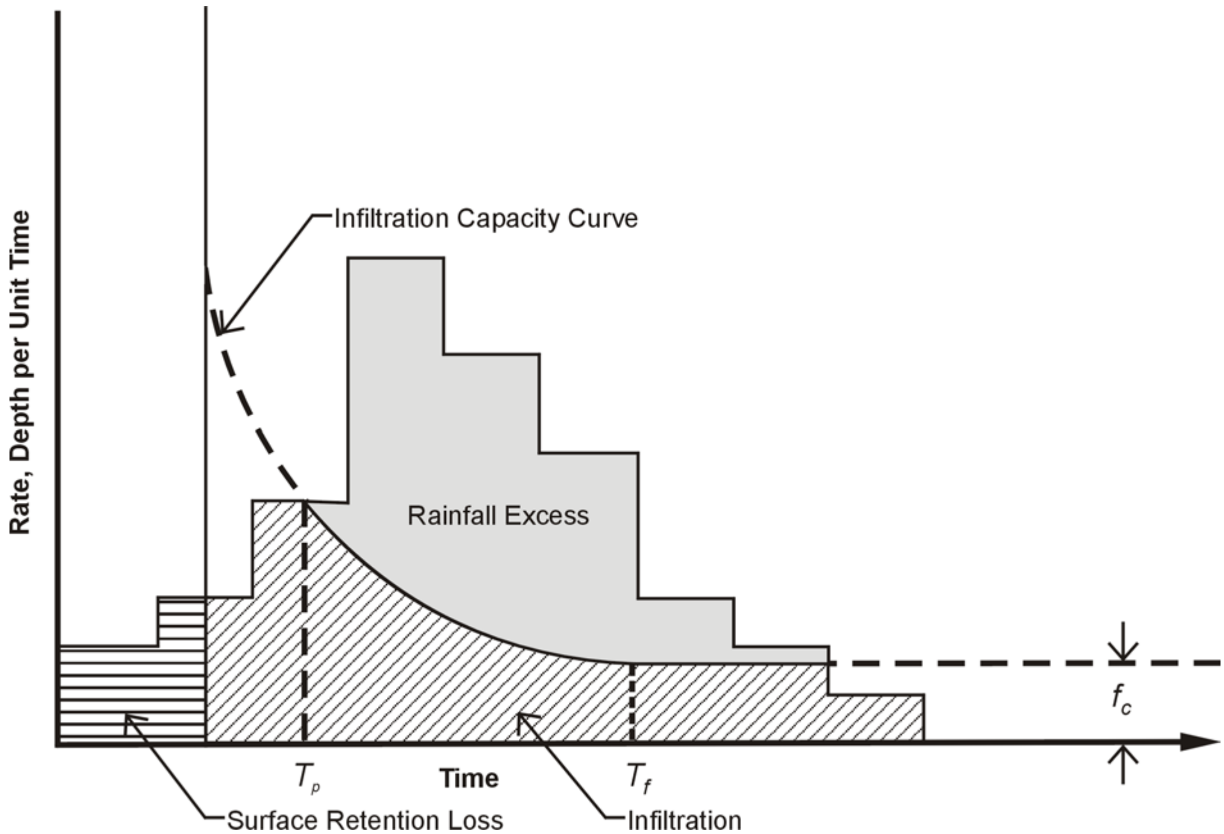
Three periods of rainfall losses are illustrated in Figure 4.1, and these must be understood and their implications appreciated before applying the procedures in this manual. First, there is a period of initial loss when no rainfall excess (runoff) is produced. During this initial period, the losses are a function of the depression storage, interception, and evaporation rates plus the initially high infiltration capacity of the soil. The accumulated rainfall loss during this period with no runoff is called the initial abstraction. The end of this initial period is noted by the onset of ponded water on the surface, and the time from start of rainfall to this time is the time of ponding (T_p). It is important to note that losses during this first period are a summation of losses due to all mechanisms including infiltration.

The second period is marked by a declining infiltration rate and generally very little losses due to other factors.

The third, and final, period occurs for rainfalls of sufficient duration for the infiltration rate to reach the steady-state, equilibrium rate of the soil (f_c). The only appreciable loss during the final period is due to infiltration.

The actual loss process is quite complex and there is a good deal of interdependence of the loss mechanisms on each other and on the rainfall itself. Therefore, simplifying assumptions are usually made in the modeling of rainfall losses. Figure 4.2 represents a simplified set of assumptions that can be made. In Figure 4.2, it is assumed that surface retention loss is the summation of all losses other than those due to infiltration, and that this loss occurs from the start of rainfall and ends when the accumulated rainfall equals the magnitude of the capacity of the surface retention loss. It is assumed that infiltration does not occur during this time. After the surface retention is satisfied, infiltration begins. If the infiltration capacity exceeds the rainfall intensity, then no rainfall excess is produced. As the infiltration capacity decreases, it may eventually equal the rainfall intensity. This would occur at the time of ponding (T_p) which signals the beginning of surface runoff. As illustrated in both Figures 4.1 and 4.2, after the time of ponding the infiltration rate decreases exponentially and may reach steady-state, equilibrium rate (f_c). It is these simplified assumptions and processes, as illustrated in Figure 4.2, that are to be modeled by the procedures in this manual.

FIGURE 4.2
SIMPLIFIED REPRESENTATION OF RAINFALL LOSSES
A FUNCTION OF SURFACE RETENTION LOSSES PLUS INFILTRATION



4.2 SURFACE RETENTION LOSS

Surface retention loss, as used herein, is the summation of all rainfall losses other than infiltration. The major component of the surface retention loss is depression storage; relatively minor components of surface retention loss are due to interception and evaporation, as previously discussed. Depression storage is considered to occur in two forms. First, in-place depression storage occurs at, and in the near vicinity of, the raindrop impact. The mechanism for this depression storage is the microrelief of the soil and soil cover. The second form of depression storage is the retention of surface runoff that occurs away from the part of the raindrop impact in surface depressions such as puddles, roadway gutters and swales, roofs, irrigation bordered fields and lawns, and so forth.

A relatively minor contribution by interception is also considered as a part of the total surface retention loss. Estimates of surface retention loss are difficult to obtain and are a function of the physiography and land-use of the area.

The surface retention loss on impervious surface has been estimated to be in the range 0.0625 inch to 0.125 inch by Tholin and Keefer (1960), 0.11 inch for 1 percent slopes to 0.06 inch for 2.5 percent slopes by Viessman (1967), and 0.04 inch based on rainfall-runoff data for an urban watershed in Albuquerque by Sabol (1983). Hicks (1944) provides estimates of surface retention losses during intense storms as 0.20 inch for sand, 0.15 inch for loam, and 0.10 inch for clay. Tholin and Keefer (1960) estimated the surface retention loss for turf to be between 0.25 and 0.50 inch. Based on rainfall simulator studies on undeveloped alluvial plains in the Albuquerque area, the surface retention loss was estimated as 0.1 to 0.2 inch (Sabol and others, 1982a). Rainfall simulator studies in New Mexico result in estimates of 0.39 inch for eastern plains range-lands and 0.09 inch for pinon-juniper hillslopes (Sabol and others, 1982b). Surface retention losses for various land-uses and surface cover conditions in Maricopa County have been extrap-olated from those reported estimates and these are shown in Table 4.2 (Section 4.4.1.3).

4.3 INFILTRATION

Infiltration is the movement of water from the land surface into the soil. Gravity and capillary forces drawing the water into and through the pore spaces of the soil matrix are the two forces that drive infiltration. Infiltration is controlled by soil properties, by vegetation influences on the soil structure, by surface cover of rock and vegetation, and by tillage practices. The distinction between infiltration and percolation is that percolation is the movement of water through the soil subsequent to infiltration.

Infiltration can be controlled by percolation if the soil does not have a sustained drainage capacity to provide access for more infiltrated water. However, before percolation can be assumed to restrict infiltration for the design rainfalls being considered in Maricopa County, the extent by which percolation can restrict infiltration of rainfall should be carefully evaluated. SCS soil scien-tists have defined hydrologic soil group D as:

“Soils having very slow infiltration rates when thoroughly wetted and consisting chiefly of clay soils with a high swelling potential, soils with a permanent high water table, soils with claypan or clay layer at or near the surface, and shallow soils over nearly impervious material.”

This definition indicates that hydrologic soil groups A, B, or C could be classified as D if a near impervious strata of clay, caliche, or rock is beneath them. When these soils are considered in regard to long-duration rainfalls (the design events for many parts of the United States) this defi-nition may be valid. However, when considered for short-duration and relatively small design rainfall depths in Maricopa County, this definition could result in underestimation of the rainfall losses. This is because even a relatively shallow horizon of soil overlaying an impervious layer still has the ability to store a significant amount of infiltrated rainfall.

For example, consider the situation where only 4 inches of soil covers an impervious layer. If the effective porosity is 0.30, then 1.2 inches (4 inches x 0.30) of water can be infiltrated and stored in the shallow soil horizon. For design rainfalls in Maricopa County, this represents a significant storage volume for infiltrated rainfall and so when developing loss rate parameters for areas of Maricopa County that contain significant areas classified as hydrologic soil group D, the reason for that classification should be determined.

Hydrologic soil group D should be retained only for:

- clay soils,
- soils with a permanent high water table, and
- rock outcrop.

Hydrologic soil group D should probably not be retained in all situations where the classification is based on shallow soils over nearly impervious layers, site specific studies and sensitivity analyses should be performed to estimate the loss rates to be used for such soils.

4.4 RECOMMENDED METHODS FOR ESTIMATING RAINFALL LOSSES

Many methods have been developed for estimating rainfall losses; five are listed as options in the HEC-1 Flood Hydrology Package. They are:

1. Holtan Infiltration Equation
2. Exponential Loss Rate
3. SCS Curve Numbers (CN) Loss Rate
4. Green and Ampt Infiltration Equation
5. Initial Loss Plus Uniform Loss Rate (IL+ULR)

Of these five, however, only the Green and Ampt and IL+ULR are recommended for estimating rainfall losses in Maricopa County for the reasons discussed below.

The Holtan Infiltration Equation is an exponential decay type of equation for which the rainfall loss rate asymptotically diminishes to the minimum infiltration rate (f_c). The Holtan equation is not extensively used and there is no known application of this method in Arizona. Data and procedures to estimate the parameters for use in Maricopa County are not available. Therefore, the Holtan equation is not recommended for general use in Maricopa County.

The Exponential Loss Rate Method is a four parameter method that is not extensively used, but it is a method preferred by the U.S. Army Corps of Engineers. Data and procedures are not available to estimate the parameters for this method for all physiographic regions in Maricopa County, but Exponential loss rate parameters have been developed from the reconstitution of flood events for a flood hydrology study in a portion of Maricopa County (U.S. Army Corps of Engineers, 1982a). However, adequate data are not available to estimate the necessary parameters for all soil types and land uses in Maricopa County, and this method is not recommended for general use in Maricopa County.

The SCS CN method previously was (pre-1990) the most extensively used rainfall loss rate method in Maricopa County and Arizona and it had wide acceptance among many agencies, consulting engineering firms, and individuals throughout the community. However, because of both theoretical concerns and practical limitations, the SCS CN method is not recommended for general use in Maricopa County.

As mentioned previously, the two recommended methods for estimating rainfall losses in Maricopa County are the Green and Ampt infiltration equation and the initial loss and uniform loss rate (IL+ULR) method. Both methods, as programmed into HEC-1, can be used to simulate the rainfall loss model as depicted in Figure 4.2. (For a full discussion of these methods, see Sections 4.4.1 and 4.4.2). The IL+ULR is a simplified model that is used extensively for flood hydrology and data often are available to estimate the two parameters for that method. The Green and Ampt infiltration equation is a physically based model that has been in existence since 1911, and is an option in HEC-1.

The preferred method, and the most theoretically accurate, is the Green and Ampt infiltration equation. That method should be used for most studies in Maricopa County where the land surface is soil, the infiltration of water is controlled by soil texture (see Appendix C, Section 5), and the bulk density of the soil is affected by vegetation. Procedures were developed, and are presented, to estimate the three parameters of the Green and Ampt infiltration equation. The alternative method of IL+ULR can be used in situations where the Green and Ampt infiltration method is recommended, but its use in those situations is not encouraged, and, in general, should be avoided. Rather, the IL+ULR method should be used in situations where the Green and Ampt infiltration equation with parameters based on soil texture is not appropriate. Examples of situations where the IL+ULR method is recommended are: large areas of rock outcrop, talus slopes, forests underlain with a thick mantle of duff, land surfaces of volcanic cinder, and surfaces that are predominantly sand and gravel. Because of the diversity of conditions that could exist for which the IL+ULR method is to be used, it is not possible to provide extensive guidance for the selection of the two parameters of the IL+ULR method.

Other methods should be used only if there is technical justification for a variance from these recommendations and if adequate information is available to estimate the necessary parameters. Use of rainfall loss methods other than those recommended should not be undertaken unless previously approved by the Flood Control District and/or the local regulatory agency.

4.4.1 Green and Ampt Infiltration Equation

This model, first developed in 1911 by W.H. Green and G.A. Ampt, has since the early 1970s, received increased interest for estimating rainfall infiltration losses. The model has the form:

$$f = K_S\left(1+\frac{\psi\theta}{F}\right) \qquad \text{for } f < i \tag{4.1}$$

$$f = i \qquad \text{for } f \geqq i$$

where:

f	=	infiltration rate (L/T),
i	=	rainfall intensity (L/T),
K_s	=	hydraulic conductivity, wetted zone, steady-state rate (L/T),
y	=	average capillary suction in the wetted zone (L),
q	=	soil moisture deficit (dimensionless), equal to effective soil porosity times the difference in final and initial volumetric soil saturations, and
F	=	depth of rainfall that has infiltrated into the soil since the beginning of rainfall (L).

A sound and concise explanation of the Green and Ampt equation is provided by Bedient and Huber (1988).

It is important to note that as rain continues, F increases and f approaches K_S, and therefore, f is inversely related to time. Equation 4.1 is implicit with respect to f which causes computational difficulties. Eggert (1976) simplified Equation 4.1 by expanding the equation in a power series and truncating all but the first two terms of the expansion. The simplified solution (Li and others, 1976) is:

$$F = -0.5\left(2F - K_S\Delta t\right) + 0.5\left[\left(2F - K_S\Delta t\right)^2 + 8K_S\Delta t\left(\theta\psi + F\right)\right]^{1/2} \tag{4.2}$$

where:

Δt	=	the computation interval, and
F	=	accumulated depth of infiltration at the start of Δt.

The average filtration rate is:

$$f = \frac{\Delta F}{\Delta t}$$

Use of the Green and Ampt equation as coded in HEC-1 involves the simulation of rainfall loss as a two phase process, as illustrated in Figure 4.2. The first phase is the simulation of the surface retention loss as previously described; this loss is called the initial loss (IA) in HEC-1. During this first phase, all rainfall is lost (zero rainfall excess generated) during the period from the start of rainfall up to the time that the accumulated rainfall equals the value of IA. It is assumed, for modeling purposes, that no infiltration of rainfall occurs during the first phase. Initial loss (IA) is primarily a function of land-use and surface cover, and recommended values of IA for use with the Green and Ampt equation are presented in Table 4.2. For example, about 0.35 inches of rainfall will be lost to runoff due to surface retention for desert and rangelands on relatively flat slopes in Maricopa County.

The second phase of the rainfall loss process is the infiltration of rainfall into the soil matrix. For modeling purposes, the infiltration begins immediately after the surface retention loss (IA) is completely satisfied, as illustrated in Figure 4.2. The three Green and Ampt equation infiltration parameters as coded in HEC-1 are:

- hydraulic conductivity at natural saturation (XKSAT) equal to K_S in equation 4.1;
- wetting front capillary suction (PSIF) equal to ψ in equation 4.1; and
- volumetric soil moisture deficit at the start of rainfall (DTHETA) equal to θ in equation 4.1.

The three infiltration parameters are functions of soil characteristics, ground surface characteristics, and land management practices. The soil characteristics of interest are particle size distribution (soil texture), organic matter, and bulk density. The primary soil surface characteristics are vegetation canopy cover, ground cover, and soil crusting. The land management practices are identified as various tillages as they result in changes in soil porosity.

Values of Green and Ampt equation parameters as a function of soil characteristics alone (bare ground condition) have been obtained from published reports (Rawls and others, 1983; Rawls and Brakensiek, 1983), and average values of XKSAT and PSIF for each of the soil texture classes are shown in columns (2) and (3) of Table 4.1. A best-fit plot of columns (2), (3), (4) and (5) is shown on Figure 4.3. Figure 4.3 should be used for selection of values of PSIF and DTHETA based on XKSAT. The values of XKSAT and PSIF from Table 4.1 or Figure 4.3 should be used if general soil texture classification of the drainage area is available. References used to create Table 4.1 can be found in the Documentation Manual.

In Table 4.1, loamy sand and sand are combined. The parameter values that are shown in the table are for loamy sand. The hydraulic conductivity (XKSAT) for sand is often used as 4.6 inches/hour, and the capillary suction (PSIF) is often used as 1.9 inches. Using those parameters values for drainage areas can result in the generation of no rainfall excess which may or may not be correct. Incorrect results could cause serious consequences for flood control planning and design. Therefore, it is recommended that, for watersheds consisting of relatively small subareas of sand, the Green and Ampt parameter values for loamy sand be used for the sand portion of the watershed. If the area contains a large portion of sand, then either the Green and Ampt method should be used with the parameter values for loamy sand or the IL+ULR method should be used with the appropriately determined values for the parameters.

TABLE 4.1
GREEN AND AMPT LOSS RATE PARAMETER VALUES FOR BARE GROUND

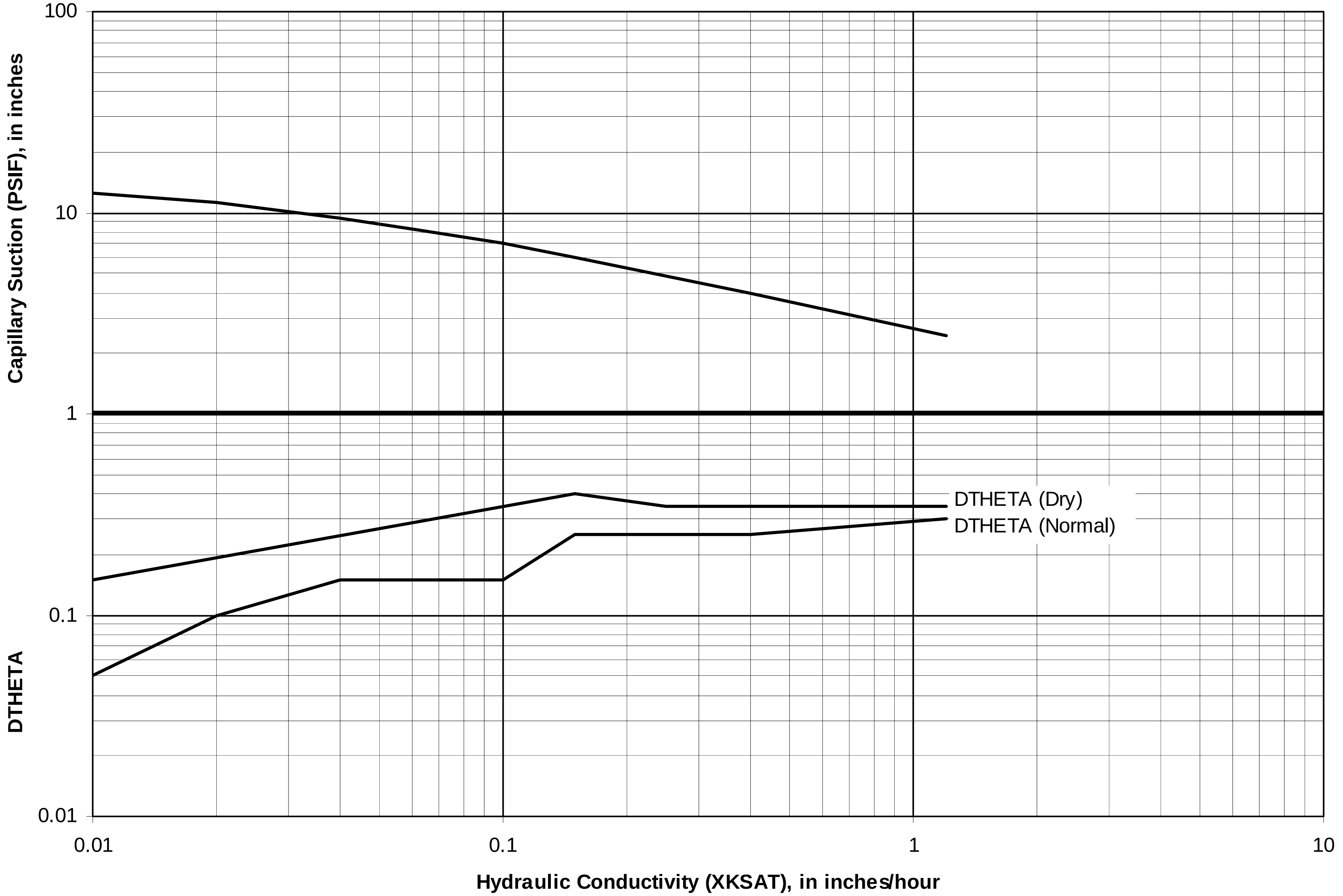
Soil Texture Classification (1)	XKSAT inches/hour (2)	PSIF inches (3)	DTHETA[1] Dry (4)	DTHETA[1] Normal (5)	DTHETA[1] Saturated (6)
loamy sand & sand	1.20	2.4	0.35	0.30	0
sandy loam	0.40	4.3	0.35	0.25	0
loam	0.25	3.5	0.35	0.25	0
silty loam	0.15	6.6	0.40	0.25	0
silt	0.10	7.5	0.35	0.15	0
sandy clay loam	0.06	8.6	0.25	0.15	0
clay loam	0.04	8.2	0.25	0.15	0
silty clay loam	0.04	10.8	0.30	0.15	0
sandy clay	0.02	9.4	0.20	0.10	0
silty clay	0.02	11.5	0.20	0.10	0
clay	0.01	12.4	0.15	0.05	0

Notes:

1. Selection of DTHETA

Dry	=	Nonirrigated lands, such as desert and rangeland;
Normal	=	Irrigated lawn, turf, and permanent pasture;
Saturated	=	Irrigated agricultural land.

FIGURE 4.3
COMPOSITE VALUES OF PSIF AND DTHETA AS A FUNCTION OF XKSAT
(TO BE USED FOR AREA-WEIGHTED AVERAGING OF GREEN AND AMPT PARAMETERS)



The soil moisture deficit (DTHETA) is a volumetric measure of the soil moisture storage capacity that is available at the start of the rainfall. DTHETA is a function of the effective porosity of the soil. The range of DTHETA is 0.0 to the effective porosity. If the soil is effectively saturated at the start of rainfall then DTHETA equals 0.0; if the soil is devoid of moisture at the start of rainfall then DTHETA equals the effective porosity of the soil.

Under natural conditions, soil seldom reaches a state of soil moisture less than the wilting point of vegetation. Due to the rapid drainage capacity of most soils in Maricopa County, at the start of a design storm the soil would not be expected to be in a state of soil moisture greater than the field capacity.

However, Maricopa County also has a large segment of its land area under irrigated agriculture, and it is reasonable to assume that the design frequency storm could occur during or shortly after certain lands have been irrigated. Therefore, it would be reasonable to assume that soil moisture for irrigated lands could be at or near effective saturation during the start of the design rainfall.

Three conditions for DTHETA have been defined for use in Maricopa County based on antecedent soil moisture condition that could be expected to exist at the start of the design rainfall. These three conditions are:

- “Dry” for antecedent soil moisture near the vegetation wilting point
- “Normal” for antecedent soil moisture condition near field capacity due to previous rainfall or irrigation applications on nonagricultural lands; and
- “Saturated” for antecedent soil moisture near effective saturation due to recent irrigation of agricultural lands.

Values of DTHETA have been estimated by subtracting the initial volumetric soil moisture for each of the three conditions from the soil porosity.

The value of DTHETA “Saturated” is always equal to 0.0 because for this condition there is no available pore space in the soil matrix at the start of rainfall. Values of DTHETA for the three antecedent soil moisture conditions are shown in Table 4.1. DTHETA “Dry” should be used for soil that is usually in a state of low soil moisture such as would occur in the desert and rangelands of Maricopa County. DTHETA “Normal” should be used for soil that is usually in a state of moderate soil moisture such as would occur in irrigated lawns, golf courses, parks, and irrigated pastures. DTHETA “Saturated” should be used for soil that can be expected to be in a state of high soil moisture such as irrigated agricultural land. However, judgement should be exercised when using a “Saturated” condition, particularly for large areas of irrigated land as it is unlikely that the entire area is being irrigated at the same time.

4.4.1.1 Procedure for Areally Averaging Green and Ampt Parameter Values

Most drainage areas or modeling subbasins will be composed of several subareas containing soils of different textures. Therefore, a composite value for the Green and Ampt parameters that are to be applied to the drainage areas for modeling subbasins needs to be determined. The procedure for determining the composite value is to average the area-weighted logarithms of the XKSAT values and to select the PSIF and DTHETA values from a graph.

The XKSAT value (and naturally occurring rock outcrop percentage) for each map unit as identified by the National Resources Conservation Service (NRCS) is provided in Appendix C. The data contained in this appendix covers the majority of the northern portion of Maricopa County. The values for XKSAT listed in the appendix are weighted based on the percentage of each unique soil texture present in the map unit and take into consideration the horizon depth of the unique soil textures in regard to the expected depth of infiltration during the design storm duration. An example of the weighting procedure along with other assumptions and criteria used in developing the XKSAT values are provided at the front of Appendix C. The composite XKSAT is calculated by Equation 4.4:

$$\overline{XKSAT} = a\log\left(\frac{\Sigma A_i \log XKSAT_i}{A_T}\right) \tag{4.4}$$

where:

$\overline{XKSAT}$	=	composite subarea hydraulic conductivity, inches/hour
$XKSAT_i$	=	hydraulic conductivity of a map unit, inches/hour (from Appendix C)
A_i	=	size of subarea
A_T	=	size of the watershed or modeling subbasin

After XKSAT is calculated, the values of PSIF and DTHETA (normal or dry) are selected from Figure 4.3, at the corresponding value of XKSAT.

4.4.1.2 Procedures for Adjusting XKSAT for Vegetation Cover

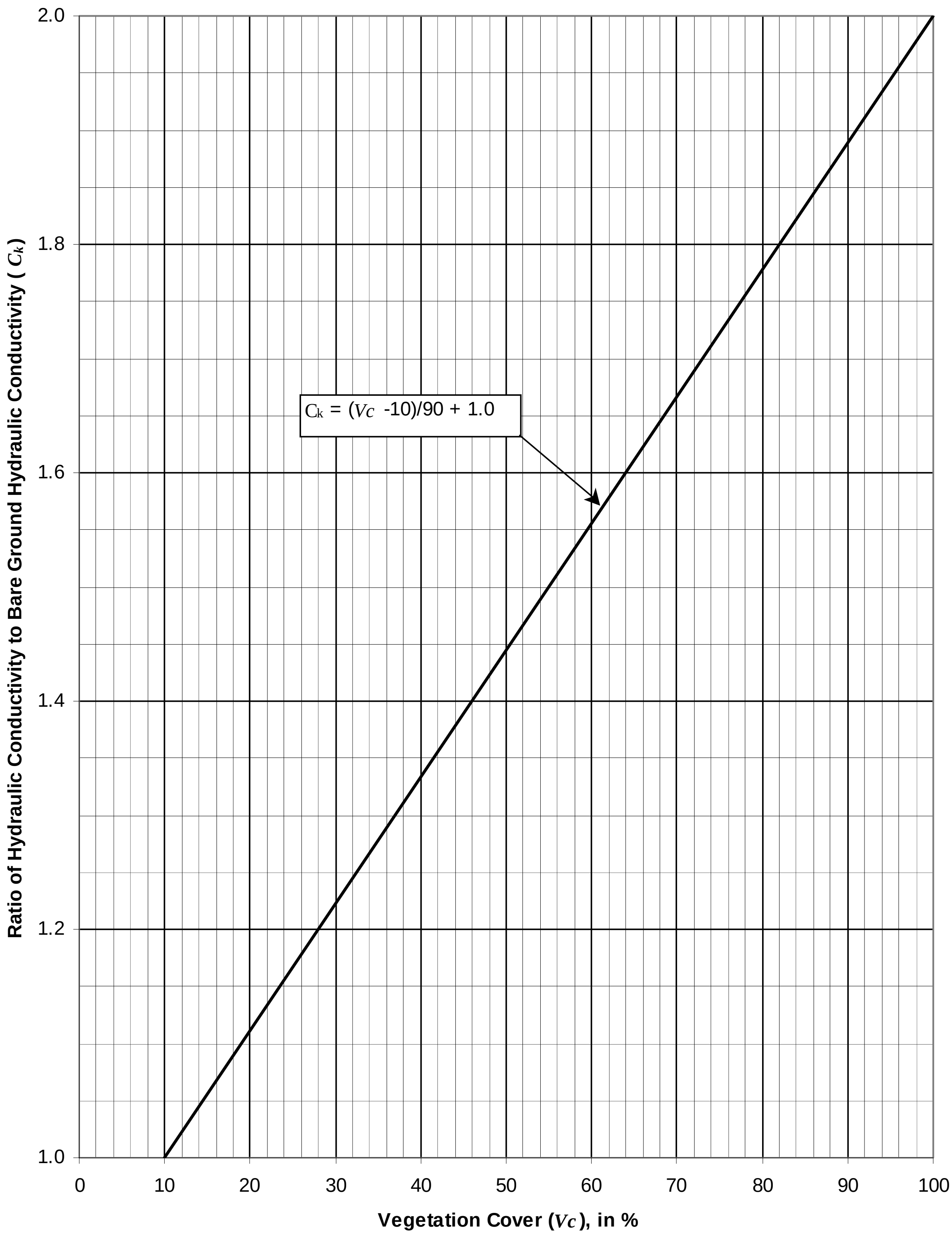
The hydraulic conductivity (XKSAT) can be affected by several factors besides soil texture. For example, hydraulic conductivity is reduced by soil crusting, increased by tillage, and increased by the influence of ground cover and canopy cover. The values of XKSAT that are presented for bare ground as a function of soil texture alone should be adjusted under certain soil cover conditions.

Ground cover, such as grass, litter, and gravel, will generally increase the infiltration rate over that of bare ground conditions. Similarly, canopy cover – such as from trees, brush, and tall grasses – can also increase the bare ground infiltration rate. The procedures and data that are presented are for estimating the Green and Ampt parameters based solely on soil texture and would be applicable for bare ground conditions. Past research has shown that the wetting front capillary suction parameter (PSIF) is relatively insensitive in comparison with the hydraulic conductivity parameter (XKSAT); therefore only the hydraulic conductivity parameter is adjusted for the influences of cover over bare ground.

Procedures have been developed (Rawls and others, 1989) for incorporating the effects of soil crusting, ground cover, and canopy cover into the estimation of hydraulic conductivity for the Green and Ampt equation; however, those procedures are not recommended for use in Maricopa County at this time. A simplified procedure to adjust the bare ground hydraulic conductivity for vegetation cover is shown in Figure 4.4. This figure is based on the documented increase in hydraulic conductivity due to various soil covers as reported by investigators using rainfall simulators on native western rangelands (Kincaid and others, 1964; Sabol and others, 1982a; Sabol and others, 1982b; Bach, 1984; Ward, 1986; Lane and others, 1987; Ward and Bolin, 1989). This correction factor can be used based on an estimate of vegetation cover as used by the NRCS in soil surveys; that is, vegetation cover is evaluated on basal area for grass and forbs, and is evaluated on canopy cover for trees and shrubs. Note that this correction can be applied only to soils other than sand and loamy sand.

The influence of tillage results in a change in total porosity and therefore a need to modify the three Green and Ampt equation infiltration parameters. The effect of tillage systems on soil porosity and the corresponding changes to hydraulic conductivity, wetting front capillary suction, and water retention is available (Rawls and Brakensiek, 1983). Although this information is available, it is not presented in this manual, nor is it recommended that these adjustments be made to the infiltration parameters for design purpose use in Maricopa County, because for most flood estimation purposes it cannot be assumed that the soil will be in any particular state of tillage at the time of storm occurrence and therefore the base condition infiltration parameters, as presented, should be used for flood estimation purposes. However, appropriate adjustment to the infiltration parameters can be made, as necessary, for special flood studies such as reconstitution of storm events.

FIGURE 4.4
EFFECT OF VEGETATION COVER ON HYDRAULIC CONDUCTIVITY FOR HYDRAULIC SOIL GROUPS B, C, AND D, AND FOR ALL SOIL TEXTURES *OTHER THAN* SAND AND LOAMY SAND



4.4.1.3 Selection of IA, RTIMP, and Percent Vegetation Cover for Urban Areas

Table 4.2 contains suggested values for IA, RTIMP, and percent vegetation cover for various natural conditions and urban land use types. The values in Table 4.2 are meant as guidelines and are not to be taken as prescribed values for these parameters. Note that the values for RTIMP reflect effective impervious areas not total impervious areas. Also, note that the values for percent vegetation cover are for pervious areas only. These three parameter values are used in the calculation of average subbasin parameters for the Green and Ampt loss method as described above. Sound engineering judgement and experience should always be used when selecting rainfall loss parameters and assigning land use categories for any given watershed.

TABLE 4.2
IA, *RTIMP*, AND VEGETATIVE COVER DENSITY FOR REPRESENTATIVE LAND USES IN MARICOPA COUNTY

Land Use[1] Code	Land Use Category	Description	IA[2] inches	RTIMP[2,3] %	Vegetation Cover[2,4] %
VLDR	Very Low Density Residential[3]	40,000 sq. feet and greater lot size	0.30	5	30
LDR	Low Density Residential[3]	12,000 – 40,000 sq. feet lot size	0.30	15	50
MDR	Medium Density Residential[3]	6,000 – 12,000 sq. feet lot size	0.25	30	50
MFR	Multiple Family Residential[3]	1,000 – 6,000 sq. feet lot size (# du/ac)	0.25	45	50
I1	Industrial 1[3]	Light and General	0.15	55	60
I2	Industrial 2[3]	General and Heavy	0.15	55	60
C1	Commercial 1[3]	Light, Neighborhood, Residential	0.10	80	75
C2	Commercial 2[3]	Central, General, Office, Intermediate	0.10	80	75
P	Pavement and Rooftops	Asphalt and Concrete, Sloped Rooftops	0.05	95	0
GR	Gravel Roadways & Shoulders	Graded and Compacted, Treated and Untreated	0.10	5	0
AG	Agricultural	Tilled Fields, Irrigated Pastures, slopes < 1%	0.50	0	85
LPC	Lawns/Parks/Cemeteries	Over 80% maintained lawn	0.20	Varies[5]	80
DL1	Desert Landscaping 1	Landscaping with impervious under treatment	0.10	95	30
DL2	Desert Landscaping 2	Landscaping without impervious under treatment	0.20	0	30
NDR	Undeveloped Desert Rangeland	Little topographic relief, slopes < 5%	0.35	Varies[5]	Varies[6]
NHS	Hillslopes, Sonoran Desert	Moderate topographic relief, slopes > 5%	0.15	Varies[5]	Varies[6]
NMT	Mountain Terrain	High topographic relief, slopes > 10%	0.25	Varies[5]	Varies[6]

Notes:
1. Other land use or zoning classifications, such as Planned Area Development and Schools must be evaluated on a case by case basis.
2. These values have been selected to fit many typical settings in Maricopa County; however, the engineer/hydrologist should always evaluate the specific circumstances in any particular watershed for hydrologic variations from these typical values.
3. RTIMP = Percent Effective Impervious Area, including right-of-way. Effective means that all impervious areas are assumed to be hydraulically connected. The RTIMP values may need to be adjusted based on an evaluation of hydraulic connectivity.
4. Vegetation Cover = Percent vegetation cover for pervious areas only.
5. RTIMP values must be estimated on a case by case basis.
6. Vegetation Cover values must be estimated on a case by case basis.

4.4.2 Initial Loss Plus Uniform Loss Rate (IL+ULR)

This is a simplified rainfall loss method that is often used, and generally accepted, for flood hydrology. In using this simplified method it is assumed that the rainfall loss process can be simulated as a two-step procedure, as illustrated in Figure 4.5. First, all rainfall is lost to runoff until the accumulated rainfall is equal to the initial loss; and second, after the initial loss is satisfied, a portion of all future rainfall is lost at a uniform rate. All of the rainfall is lost if the rainfall intensity is less than the uniform loss rate.

According to HEC-1 nomenclature, two parameters are needed to use this method; the initial loss (STRTL) and the uniform loss rate (CNSTL).

Because this method is to be used for special cases where infiltration is not controlled by soil texture, or for drainage areas and subbasins that are predominantly sand, the estimation of the parameters will require model calibration, results of regional studies, or other valid techniques. It is not possible to provide complete guidance in the selection of these parameters; however, some general guidance is provided:

A. For special cases of anticipated application, the uniform loss rate (CNSTL) will either be very low for nearly impervious surfaces, or possibly quite high for exceptionally fast-draining (highly pervious) land surfaces. For land surfaces with very low infiltration rates, the value of CNSTL will probably be 0.05 inches per hour or less. For sand, a CNSTL of 0.5 to 1.0 inch per hour or larger may be reasonable. Higher values of CNSTL for sand and other surfaces are possible, however, use of high values of CNSTL would require special studies to substantiate the use of such values.

B. Although the IL+ULR method is not recommended for watersheds where the soil textures can be defined and where the Green and Ampt method is encouraged, some general guidance in the selection of the uniform loss rate is shown in Tables 4.3 and 4.4. Table 4.4 was prepared based on the values in Table 4.3 and the hydraulic conductivity's shown in Table 4.1. In Table 4.4, the initial infiltration (II) is an estimate of the infiltration loss that can be expected prior to the generation of surface runoff. The value of initial loss (STRTL) is the sum of initial infiltration (II) of Table 4.4 and surface retention loss (IA) of Table 4.2; STRTL = II + IA.

C. The estimation of initial loss (STRTL) can be made on the basis of calibration or special studies at the same time that CNSTL is estimated. Alternatively, since STRTL is equivalent to initial abstraction, STRTL can be estimated by use of the SCS CN equations for estimated initial abstraction, written as:

$$STRTL = \frac{200}{CN} - 2 \tag{4.5}$$

Estimates for CN for the drainage area or subbasin should be made referring to various publications of the SCS, particularly TR-55. Equation 4.5 should provide a fairly good estimate of STRTL in many cases, however, its use should be judiciously applied and carefully considered in all cases.

FIGURE 4.5
REPRESENTATION OF RAINFALL LOSS ACCORDING TO THE INITIAL LOSS PLUS UNIFORM LOSS RATE (IL + ULR)

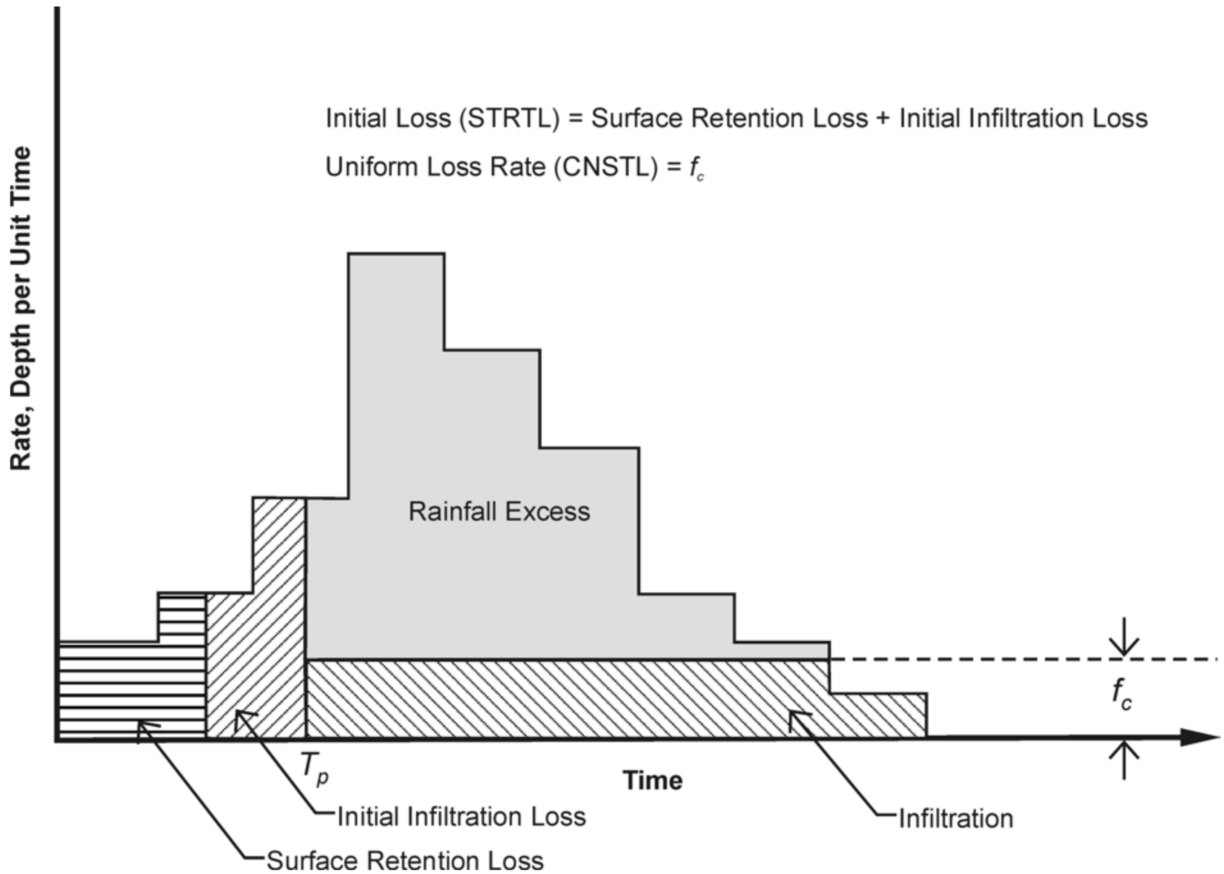


TABLE 4.3
PUBLISHED VALUES OF UNIFORM LOSS RATES

Hydrologic Soil Group (1)	Uniform Loss Rate, inches/hour		
	Musgrave (1955) (2)	USBR (1975)[1] (3)	USBR (1987)[2] (4)
A	0.30 – 0.45	0.40	0.30 – 0.50
B	0.15 – 0.30	0.24	0.15 – 0.30
C	0.05 – 0.15	0.12	0.05 – 0.15
D	0 – 0.05	0.08	0 – 0.05

Notes:
1. Design of Small Dams, Second Edition, 1975, Appendix A.
2. Design of Small Dams, Third Edition, 1987.

TABLE 4.4
INITIAL LOSS PLUS UNIFORM LOSS RATE PARAMETER VALUES FOR BARE GROUND ACCORDING TO HYDROLOGIC SOIL GROUP

Hydrologic Soil Group (1)	Uniform Loss Rate CNSTL (2)	Initial Infiltration, inches II[1]		
		Dry (3)	Normal (4)	Saturated (5)
A	0.4	0.6	0.5	0
B	0.25	0.5	0.3	0
C	0.15	0.5	0.3	0
D	0.05	0.4	0.2	0

Notes:

1. Selection of II:

Dry	=	Nonirrigated lands, such as desert and rangeland;
Normal	=	Irrigated lawn, turf, and permanent pasture;
Saturated	=	Irrigated agricultural land.

4.5 PROCEDURE FOR ESTIMATING LOSS RATES

Procedures for estimating rainfall loss rates are provided in the following sections. Notes and general guidance on the application of these procedures are provided along with a detailed example using the Green and Ampt method in Chapter 9, Section 9.3.

4.5.1 Green and Ampt Method

A. When soils data are available:

1. Prepare a base map of the drainage area delineating subbasins, if used.

2. Determine the location of the study area in regard to the limits of the soil surveys provided in Appendix C.

    a. If the study area is completely contained within these limits:

        i. Overlay the watershed limits on the soil survey maps from the appropriate soil survey report(s) and tabulate the map units present within the watershed.

        ii. Cross reference the map units with those listed in Appendix C and tabulate the weighted value of XKSAT for each map unit and the corresponding percent imperviousness.

        iii. Proceed to item (3) or (4).

b. If the study area is partly or entirely outside the limits of the soils surveys provided in Appendix C:

   i. Refer to the figure showing the status of soil surveys in Arizona (at the front of Appendix C) for other sources of soils data. Other sources of soils data are:

      - General soils surveys by county prepared by the NRCS.
      - Other detailed soil surveys.
      - Terrestrial Ecosystem Survey of Tonto National Forest.

   ii. Using the data contained in the alternative source, follow the example procedure for determination of the weighted XKSAT value for each unique map unit that is included at the front of Appendix C.

   iii. Proceed to item (3) or (4).

3. If the watershed or subbasin contains only one soil texture, then use Figure 4.3 to select the value of PSIF and DTHETA.

4. If the watershed or subbasin is composed of soils of different textures, then area-weighted parameter values will be calculated:

   a. Calculate the area-weighted value of XKSAT by using Equation 4.4.

   b. Select the corresponding values of PSIF and DTHETA from Figure 4.3.

   c. Calculate the arithmetically area-weighted value of naturally occurring RTIMP.

5. Select values of IA for each land use and/or soil cover using Table 4.2. Arithmetically area-weight the values of IA if the drainage area or subbasin is composed of subareas of different IA.

6. Select values of RTIMP for each land use using Table 4.2. Arithmetically area-weight the values of RTIMP if the drainage area or subbasin is composed of land use subareas of different RTIMP. Compute the total weighted value of RTIMP based on the area-weighted land use and naturally occurring RTIMP.

7. Estimate the vegetative cover (VC) for the natural portions of the drainage area or subbasin. Select values of VC for each land use using Table 4.2. Arithmetically area-weight the values of VC if the drainage area or subbasin is composed of land use

subareas of different VC. Arithmetically average the natural VC and the area-weighted land use VC.

8. Adjust the XKSAT value for VC using Figure 4.4, if appropriate.

9. Arithmetically average $DTHETA_{dry}$ (natural portions of the drainage area or subbasin) and $DTHETA_{normal}$ (Developed portions of the drainage area or subbasin), if appropriate.

B. Alternative Methods:

As an alternative to the above procedures, Green and Ampt loss rate parameters can be estimated by reconstitution of recorded rainfall-runoff events on the drainage area or hydrologically similar watersheds, or parameters can be estimated by use of rainfall simulators in field experiments. Plans and procedures for estimating Green and Ampt loss rate parameters by either of these procedures should be approved by the Flood Control District and/or the local agency before initiating the procedures.

4.5.2 Initial Loss Plus Uniform Loss Rate Method

A. When soils data are available:

1. Prepare a base map of the drainage area delineating modeling subbasins, if used.

2. Delineate subareas of different infiltration rates (uniform loss rates) on the base map. Assign a land-use or surface cover to each subarea.

3. Determine the size of each subbasin and size of each subarea within each subbasin.

4. Estimate the impervious area (RTIMP) for the drainage area or each subarea.

5. Estimate the initial loss (STRTL) for the drainage area or each subarea by regional studies or calibration. Alternatively, Equation 4.5 or Tables 4.2 and 4.4 can be used to estimate or to check the value of STRTL.

6. Estimate the uniform loss rate (CNSTL) for the drainage area or each subarea by regional studies or calibration. Table 4.3 can be used, in certain situations, to estimate or to check the values of CNSTL.

7. Calculate the area-weighted values of RTIMP, STRTL, and CNSTL for the drainage area or each subbasin.

8. Enter the area-weighted values of RTIMP, STRTL, and CNSTL for the drainage area or each subbasin on the LU record of the HEC-1 input file.

5 UNIT HYDROGRAPH PROCEDURES

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5.1 GENERAL

Rainfall excess can be routed from a watershed to produce a storm discharge hydrograph at a downstream location (concentration point) by one of two methods: 1) hydraulic routing involving the complete or some simplified form of the equations of motion (i.e., the momentum equation plus the continuity equation); or 2) hydrologic routing involving the application of the continuity equation. Kinematic wave routing, as available in HEC-1, is an example of simplified hydraulic routing. Hydrologic routing is usually accomplished by either direct application of the equation of continuity (Equation 5.1), or a graphical procedure such as the application of the principles of the unit hydrograph.

$$I - O = \frac{dS}{dt} \tag{5.1}$$

Examples of hydrologic routing by direct application of the equation of continuity are the Clark Unit Hydrograph (Clark, 1945), the Santa Barbara Urban Hydrograph (Stubchaer, 1975), and the Single Linear Reservoir Model (Pedersen and others, 1980). Both the Santa Barbara Urban Hydrograph and the Single Linear Reservoir Model are simplified (one parameter) versions of the

Clark Unit Hydrograph (three parameter) procedure (Sabol and Ward, 1985). Examples of unit hydrographs that require a graphical procedure are the SCS Dimensionless Unit Hydrograph, Snyder's Unit Hydrograph, S-graphs, and unit hydrographs that are derived directly from recorded runoff data. Graphical or tabular methods of routing rainfall excess by unit hydrographs are very amenable to hand-calculation methods commonly used before computers became readily available. Direct mathematical solution of the equation of continuity, such as the Clark Unit Hydrograph, is more efficiently conducted with computers and appropriate computer programs.

The recommended procedures for routing rainfall excess in Maricopa County are either the Clark Unit Hydrograph or the application of selected S-graphs. The Clark Unit Hydrograph procedure, as described herein, is recommended for watersheds or subbasins less than about 5 square miles in size with an upper limit of application of 10 square miles and is the preferred procedure for urban watersheds. The application of S-graphs is recommended for use with major watercourses in Maricopa County.

A unit hydrograph is a graph of the time distribution of runoff from a specific watershed as the result of one inch of rainfall excess that is distributed uniformly over the watershed and that is produced during a specified time period (duration). The duration of rainfall excess is not generally equal to the rainfall duration. A unit hydrograph is derived from or is representative of a specific watershed; therefore, a unit hydrograph is a lumped parameter that reflects all of the physical characteristics of the watershed that affect the time rate at which rainfall excess drains from the land surface.

The principles of the unit hydrograph were introduced by Sherman (1932) who observed that for a watershed all hydrographs resulting from a rain of the same duration have the same time base, and that ordinates of each storm hydrograph from the watershed are proportional to the volume of runoff if the time and areal distributions of the rainfalls are similar. The principles that are applied when using a unit hydrograph are:

1. For a watershed, hydrograph base lengths are equal for rainfall excesses of equal duration.

2. Hydrograph ordinates are proportional to the amount of rainfall excess.

3. A storm hydrograph can be developed by linear superposition of incremental hydrographs.

Application of these principles requires a linear relation between watershed outflow and storage within the watershed, $S = KO$. However, Mitchell (1962) has shown that nonlinear storage, $S = KO^x$, is a condition that occasionally occurs in natural watersheds. A method has been developed by Shen (1962) to evaluate the linearity of the storage-outflow relation for gaged watersheds. Mitchell (1972) developed the model hydrograph for use in watersheds that have nonlinear storage-outflow characteristics. Presently no method has been devised to evaluate the linearity of an ungaged watershed, and the assumption of linearity is a practical necessity in virtually all cases.

5.2 CLARK UNIT HYDROGRAPH

Hydrologic routing by the Clark Unit Hydrograph method is analogous to the routing of an inflow hydrograph though a reservoir. This analogy is illustrated in Figure 5.1. The inflow hydrograph, called the translation hydrograph in the Clark method, is determined from the temporal and spatial distribution of rainfall excess over the watershed. The translation hydrograph is then routed by a form of the equation of continuity:

$$O_i = CI_i + (1 - C)\, O_{i-1} \tag{5.2}$$

$$C = \frac{2\Delta t}{2R + \Delta t} \tag{5.3}$$

O_i is the instantaneous flow at the end of the time period; O_{i-1} is the instantaneous flow at the beginning of the time period; I_i is the ordinate of the translation hydrograph; Δt is the computation time interval; and R is the watershed storage coefficient. The Clark Unit Hydrograph of duration, Δt, is obtained by averaging two instantaneous unit hydrographs spaced Δt units apart:

$$U_i = 0.5(O_i + O_{i-1}) \tag{5.4}$$

where:

U_i = the ordinates of the Clark Unit Hydrograph.

The Clark method uses two numeric parameters, T_c and R, and a graphical parameter, the time-area relation. Clark (1945) defined T_c as the time from the end of effective rainfall over the watershed to the inflection point on the recession limb of the surface runoff hydrograph as shown in Figure 5.2. In practice, for ungaged watersheds this time is usually estimated by empirical equations since runoff hydrographs from the watershed are not often available.

The second parameter is the storage coefficient, R, which has the dimension of time. This parameter is used to account for the effect that temporary storage in the watershed has on the hydrograph. Several methods are available to estimate R from recorded hydrographs for a basin. As originally proposed by Clark (1945), this parameter can be estimated by dividing the discharge at the point of inflection of the surface runoff hydrograph by the rate of change of discharge (slope of the hydrograph) at the inflection point as shown in Figure 5.2.

Another technique for estimating R is to compute the volume remaining under the recession limb of the surface runoff hydrograph following the point of inflection and to divide the volume by the discharge at the point of inflection. Both of these methods require the ability to identify the inflection point on the recession limb of the runoff hydrograph. This is difficult if not impossible for complex hydrographs and flashy hydrographs such as occur from urban basins and natural watersheds in the Southwest. A method to estimate R by a graphical recession analysis of the hydrograph has been proposed (Sabol, 1988) and this method provides much more consistent results than do the previously described methods. The parameter, R, should be estimated by the analysis of several recorded events; however, in most cases recorded discharge hydrographs are not available and R must be estimated by empirical equations.

The time-area relation, a graphical parameter, is necessary to compute the translation hydrograph. The time-area relation specifies the accumulated area of the watershed that is contributing runoff to the outlet of the watershed at any point in time. Procedures to develop a time-area relation for a watershed are discussed in a later section of this manual.

FIGURE 5.1
CONCEPTUAL ANALOGY OF LINEAR RESERVOIR ROUTING TO THE GENERATION OF A STORM HYDROGRAPH BY THE CLARK UNIT HYDROGRAPH METHOD

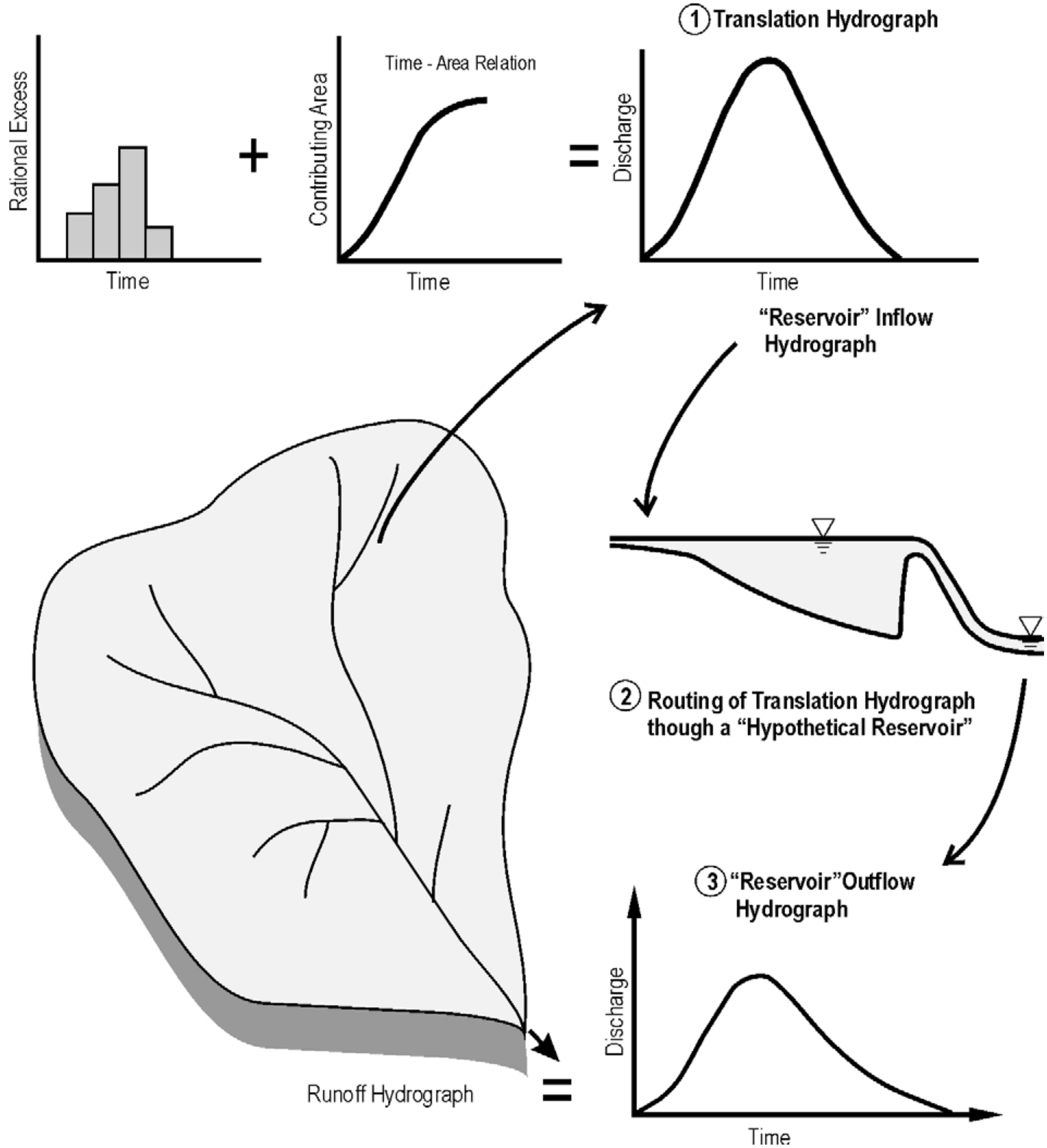
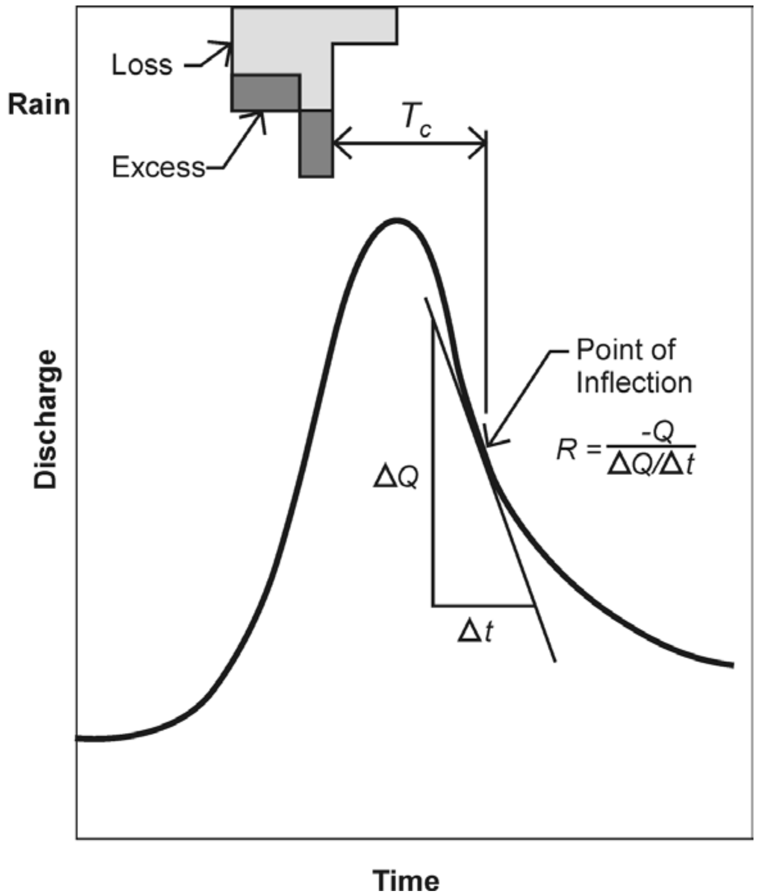


FIGURE 5.2
DEFINITION SKETCH OF CLARK UNIT HYDROGRAPH PARAMETERS FROM HYDROGRAPH ANALYSIS



The application of the Clark Unit Hydrograph method is best described with a simple example. A watershed is shown in Figure 5.3(a), and a rainfall hyetograph and rainfall excess distribution area shown in Figure 5.3(b). For the example watershed and given intensity of rainfall excess, the time of concentration is estimated at 25 minutes. An isochrone interval of 5 minutes is selected and the watershed is divided into five zones by isochrones as shown in Figure 5.3(a). The areas within each isochrone zone are measured and the dimensionless time-area relation is developed as shown in the table and depicted in Figure 5.3(c). The translation hydrograph of the time rate of runoff is developed by considering each incremental unit of runoff production that would be available as inflow to a watershed routing model. For example, at the end of the first 5 minutes of rainfall excess the runoff that is available at the outlet of the watershed is the product of incremental area A_1, and rainfall excess R_1.

$$I_1 = (A_1 R_1) \times \frac{c}{\Delta t}$$

where:

c = 60.5 cfs/acre-inch/minute

Δt = 5 minutes

I_1 = (8 acres)(0.10 inch)(60.5 cfs/acre-inch/minute)/(5 minutes)

= 9.7 cfs

<u>At the end of 10 minutes the available runoff is:</u>

= 82.3 cfs

<u>At the end of 15 minutes the available runoff is:</u>

= 234.7 cfs

<u>At the end of 20 minutes the available runoff is:</u>

= 393.5 cfs

<u>At the end of 25 minutes the available runoff is:</u>

= 416.2 cfs

Notice that, for this example, all incremental rainfalls equal 0.0 from R_5 onward.

<u>At the end of 30 minutes the available runoff is:</u>

$$I_6 = (A_3R_4 + A_4R_3 + A_5R_2)\times\frac{c}{\Delta t}$$

$$= \left[(38)(.15) + (32)(.30) + (18)(.55)\right]\times\frac{60.5}{5}$$

= 304.9 cfs

<u>At the end of 35 minutes the available runoff is:</u>

$$I_7 = (A_4R_4 + A_5R_3)\times\frac{c}{\Delta t}$$

$$= \left[(32)(.15) + (18)(.30)\right]\times\frac{60.5}{5}$$

= 123.4 cfs

<u>At the end of 40 minutes the available runoff is:</u>

$$I_8 = (A_5R_4)\times\frac{c}{\Delta t}$$

$$= \left[(18)(.15)\right]\times\frac{60.5}{5}$$

= 32.7 cfs

<u>After 45 minutes (rainfall excess of 20 minutes plus travel time of 25 minutes) the available runoff is:</u>

I_9 = 0 cfs

The translation hydrograph (I_i) is shown in Figure 5.3(d). This theoretical hydrograph has the correct volume of runoff from the watershed, however it does not reflect the effects of routing through the watershed. The translation hydrograph is then routed and averaged using Equation 5.2 through 5.4 resulting in the final runoff hydrograph. For example, assume that R = 15 minutes, and the runoff hydrograph is shown in Figure 5.3(d).

TABLE 5.1
RUNOFF HYDROGRAPH

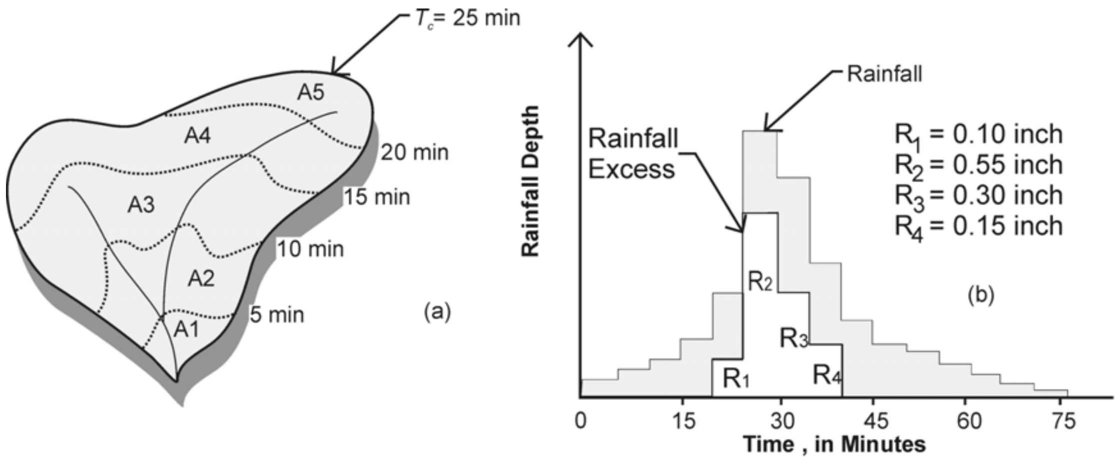
		Hydrograph		
	Time	Translation, (*I*)	Instantaneous, (*O*)	Runoff, (*U*)
Increment	minutes	cfs	cfs	cfs
(1)	(2)	(3)	(4)	(5)
1	5	9.7	2.8	1.4
2	10	82.3	25.9	14.3
3	15	234.7	86.4	56.1
4	20	393.5	175.5	131.0
5	25	416.2	245.3	210.4
6	30	304.9	262.6	253.9
7	35	123.4	222.2	242.4
8	40	32.7	167.3	194.7
9	45	0.0	118.8	143.0
10	50	0.0	84.3	101.5
11	55	0.0	59.9	72.1
12	60	0.0	42.5	51.2
13	65	0.0	30.2	36.3
14	70	0.0	21.4	25.8

Notes:

1. Δt = 5 minutes
2. R = 15 minutes
3. $C = 2\Delta t/(2R + \Delta t) = 0.29$
4. Assume O_{i-1} for increment 1 = 0.0

Notice that the Clark Unit Hydrograph itself was never developed per se but that the three principles of the unit hydrograph were applied directly (mathematically) to the rainfall excess without performing graphical superposition of ratios of a unit hydrograph. Computationally, this process can be completed very quickly and conveniently with a computer program such as is done with HEC-1.

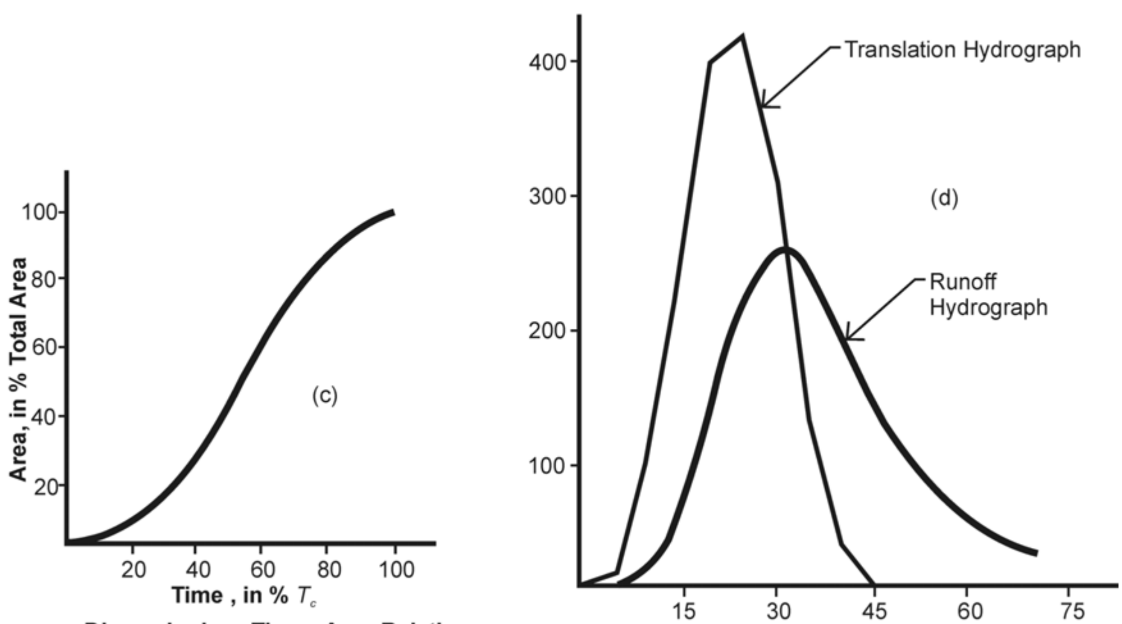
FIGURE 5.3
EXAMPLE OF STORM HYDROGRAPH GENERATION USING THE CLARK UNIT HYDROGRAPH METHOD



Watershed Map and Isochrones **Rainfall Hydrograph and Rainfall Excess Distribution**

Table Showing Development of Dimensionless Time-Area Relation

Isochrone Zone	Area Acres	Accumulated Area	Accumulated Area as % of Total Area	Travel Time as % of *Tc*
(1)	(2)	(3)	(4)	(5)
A_1	8	8	6.7	20
A_2	24	32	26.7	40
A_3	38	70	58.3	60
A_4	32	102	85	80
A_5	18	120	100	100



Dimensionless Time - Area Relation **Time Since Start of Rainfall Excess**

5.3 LIMITATIONS AND APPLICATIONS

There are no theoretical limitations governing the application of the Clark Unit Hydrograph; however, there are some practical limitations that should be observed. The method that is used to estimate the parameters may dictate limitations in regard to the type or size of watershed that is being considered. If the parameters are estimated through an analysis or reconstitution of a recorded rainfall-runoff event, the parameters would be considered to be appropriate for that particular watershed, regardless of type or size. This is the preferred method of parameter estimation, but there will be limited opportunity for this approach because of the scarcity of instrumented watersheds in Maricopa County. The parameters could be estimated by indirect methods, such as regional analysis of recorded data. In this case, application of the parameter estimation procedures should be applied only to those ungaged watersheds that are representative of the watersheds in the database. Most often, the parameters are estimated by generalized relations that may have been developed from a relatively large and diverse database. The parameter estimation procedures that are recommended herein are of the last category.

The Clark Unit Hydrograph parameter estimation procedures that are presented in this manual have been adopted, modified, or developed from an analysis of a large data base of instrumented watersheds, controlled experimental watersheds, and laboratory studies; therefore, the application of these procedures is considered to be appropriate for most conditions that occur in Maricopa County. The types of watersheds for which the procedures can be applied include urban, rangeland, alluvial fans, agricultural, hillslopes, and mountains.

Watershed size should be 5 square miles or less, with an upper limit of application to a single basin of 10 square miles. Watersheds larger than 5 square miles should be divided into smaller sub-basins for modeling purposes. Many watersheds smaller than 5 square miles should also be divided into sub-basins depending on the drainage network and degree of homogeneity of the watershed. The subdivision of the watershed into near homogeneous units should result in improved accuracy. Subdivision may also be desirable or required to determine discharges at concentration points within the watershed.

5.4 DEVELOPMENT OF PARAMETER ESTIMATORS

The procedures for parameter estimation are based on available literature, research results, and analysis of original data. For example, the T_c equation is based on the research of Papadakis and Kazan (1987). A large database of recorded rainfall-runoff data was compiled and analyzed in developing and testing the procedures. These data are for instrumented watersheds in Arizona, New Mexico, Colorado, and Wyoming. A discussion of the development and testing of these procedures is contained in the Documentation Manual that is a companion to the *Hydrology Manual*.

5.5 ESTIMATION OF PARAMETERS

The following procedures are recommended for the calculation of the Clark Unit Hydrograph parameters for use in Maricopa County. Other general procedures, as previously discussed, can be used; however, those should be approved by the jurisdictional agency prior to undertaking such procedures.

5.5.1 Time of Concentration

Time of concentration is defined as the travel time, during the corresponding period of most intense rainfall excess, for a floodwave to travel from the hydraulically most distant point in the watershed to the point of interest (concentration point). Note especially that T_c is not the travel time taken for a particle of water to move down the catchment, as is often cited in engineering texts. The catchment is in equilibrium when T_c is reached because the outlet then “feels” the inflow from every portion of the catchment (Bedient and Huber, 1988). Since a wave moves faster than a particle of water, the time of concentration (and catchment equilibrium) occurs sooner than if based on overland flow or channel water velocities. An empirical equation for time of concentration, T_c has been adopted with some procedural modifications from Papadakis and Kazan (1987).

$$T_c = 11.4\, L^{0.50}\, K_b^{0.52}\, S^{-0.31} i^{-0.38} \qquad (5.5)$$

where:

T_c	=	time of concentration, in hours.
L	=	length of the longest flow path, in miles.
K_b	=	watershed resistance coefficient (see Figure 5.5, or Table 5.3).
S	=	watercourse slope, in feet/mile.
i	=	the average rainfall excess intensity, during the time T_c, in inches/hour.

Watercourse slope S is the average slope of the flow path for the same watercourse that is used to define L. The magnitude of S can be calculated as the difference in elevation between the two points used to define L divided by the length, L. Watersheds in mountains can result in large values for S, which may result in an underestimation of T_c. This is because as slope increases in natural watersheds the runoff velocity does not usually increase in a corresponding manner. The slope of steep natural watercourses is often adjusted to reduce the slope, and the reduced slope of steep natural watercourses should be adjusted by using Table 5.2 or Figure 5.4.

FIGURE 5.4
SLOPE ADJUSTMENT FOR STEEP WATERCOURSES IN NATURAL WATERSHEDS
(Source: Drainage Criteria Manual, Urban Drainage and Flood Control District, Colorado, May 1984.)

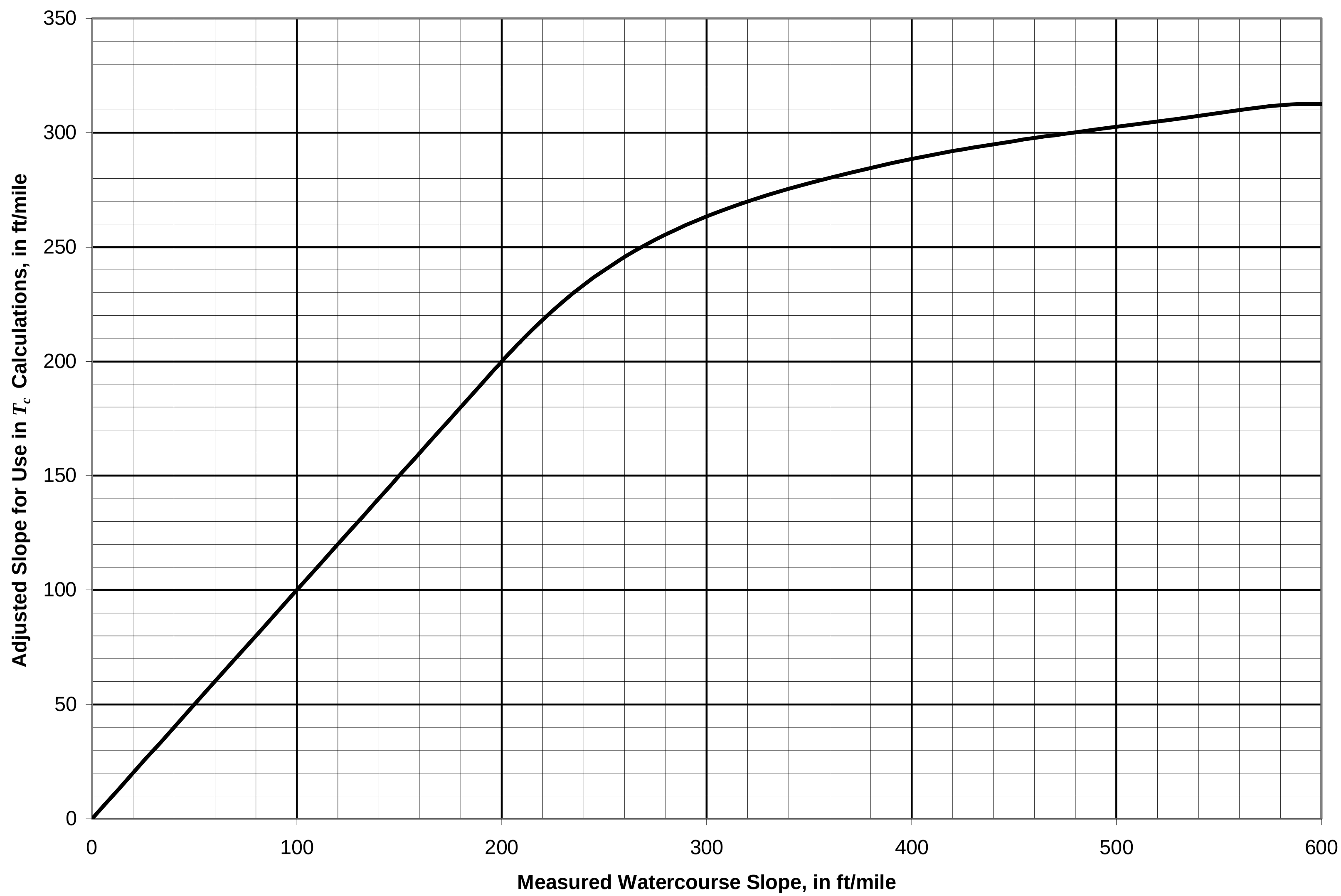


TABLE 5.2
SLOPE ADJUSTMENT FOR STEEP WATERCOURSES

Natural Slope (S)	Adjusted Slope (S_{adj})	Natural Slope (S)	Adjusted Slope (S_{adj})
200	200	410	290
210	209	420	292
220	218	430	294
230	226	440	295
240	233	450	296
250	240	460	298
260	246	470	299
270	251	480	300
280	255	490	301
290	260	500	303
300	263	510	304
310	267	520	305
320	270	530	306
330	273	540	307
340	275	550	309
350	278	560	310
360	280	570	311
370	283	580	312
380	285	590	313
390	287	600	313
400	288		

The adjusted slope is based on the following:

1. For $0 < S <= 200$, $S_{adj} = S$
2. For $200 < S <= 600$, $S_{adj} = a_0+a_1S+a_2S^2+a_3S^3+a_4S^4+a_5S^5+a_6S^6+a_7S^7$

   where:

   $a_0 =$ 6.725897827E+02
   $a_1 =$ -1.634093666E+01
   $a_2 =$ 1.739404649E-01
   $a_3 =$ -8.902683621E-04
   $a_4 =$ 2.552852266E-06
   $a_5 =$ -4.203532411E-09
   $a_6 =$ 3.721179614E-12
   $a_7 =$ -1.374400319E-15

The selection of a representative watershed resistance coefficient, K_b, similar in concept to Manning's n in open-channel flow, is very subjective and therefore a high degree of uncertainty is associated with its use. To diminish this uncertainty and to increase the reproducibility of the procedure, a graph is provided in Figure 5.5 for the selection on K_b based on watershed classification and watershed size. Interpolation can be used for a given watershed size and mixed classification. Equations for estimating K_b are given in Table 5.2, along with general descriptions of land forms/use for which the equation applies.

The value of i in Equation 5.5 requires the knowledge of both the distribution of rainfall excess intensity and the time of concentration, which is of course, unknown. Therefore, Equation 5.5 must be solved in a trial and error procedure. First, the time distribution of rainfall excess must be estimated for the design rainfall distribution and a graph of average rainfall excess intensity versus time prepared. Then a value of T_c is assumed and the corresponding value of i is read from the graph. Equation 5.5 is solved with that value of i. If the calculated value of T_c is reasonably close to the value that was assumed for i then the solution is finished; if not, then assume a new value of T_c, recalculate i, and recalculate T_c with Equation 5.5. The solution for T_c should converge within three trials.

A worksheet has been prepared that facilitates the calculation of T_c. Appendix D, Section 1 is a copy of this worksheet and Section 5.9 of this manual shows how it is used. Alternatively, the DDMSW program can be used, which will also populate the HEC-1 input file with the required data.

The computation interval (NMIN) on the IT record of HEC-1 must be selected to correspond to the time of concentration for the unit hydrograph. This requirement is necessary to adequately define the shape of the unit hydrograph. From Snyder's unit hydrograph theory, the unit rainfall duration for a unit hydrograph (computation interval) is equal to lag time divided by 5.5. For the SCS Dimensionless Unit Hydrograph, the unit rainfall duration is to equal 0.133 T_c, and although small variation in the selection of computation interval is allowed, the SCS recommends that the duration not exceed 0.25 T_c. Although there is not a rigid theoretical limitation to how small the computation interval can be, from a practical standpoint, too small of a NMIN could result in excessive computer output. Therefore, as a general rule the computation interval should meet the following:

$$\text{NMIN} = 0.15\ T_c \qquad (5.6)$$

Equation 5.6 is preferred, however, as a general requirement NMIN should fall in the range indicated in Equation 5.7.

$$0.10\ T_c < \text{NMIN} < 0.25\ T_c \qquad (5.7)$$

FIGURE 5.5

RESISTANCE COEFFICIENT K_b AS A FUNCTION OF WATERSHED SIZE AND SURFACE ROUGHNESS CHARACTERISTICS

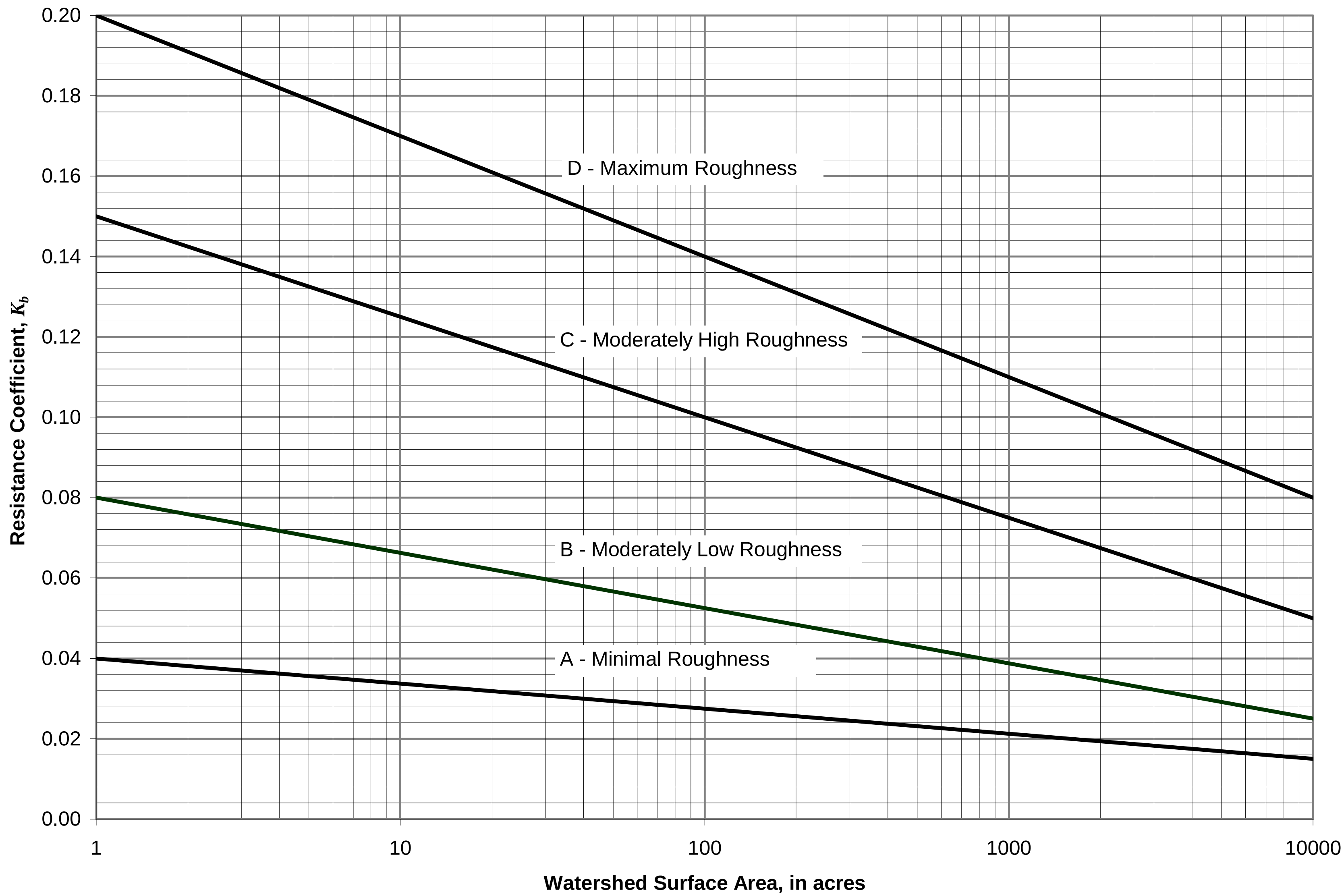


TABLE 5.3
EQUATION FOR ESTIMATING K_b IN THE T_c EQUATION

$K_b = m \log A + b$ Where A is drainage area, in acres				
			Equation Parameters	
Type	**Description**	**Typical Applications**	***m***	***b***
A	Minimal roughness: Relatively smooth and/or well graded and uniform land surfaces. Surfaces runoff is sheet flow.	Commercial/industrial areas Residential area Parks and golf courses	-0.00625	0.04
B	Moderately low roughness: Land surfaces have irregularly spaced roughness elements that protrude from the surface but the overall character of the surface is relatively uniform. Surface runoff is predominately sheet flow around the roughness elements.	Agricultural fields Pastures Desert rangelands Undeveloped urban lands	-0.01375	0.08
C	Moderately high roughness: Land surfaces that have significant large to medium-sized roughness elements and/or poorly graded land surfaces that cause the flow to be diverted around the roughness elements. Surface runoff is sheet flow for short distances draining into meandering drainage paths.	Hillslopes Brushy alluvial fans Hilly rangeland Disturbed land, mining, etc. Forests with underbrush	-0.025	0.15
D	Maximum roughness: Rough land surfaces with torturous flow paths. Surface runoff is concentrated in numerous short flow paths that are often oblique to the main flow direction.	Mountains Some wetlands	-0.030	0.20

5.5.2 Storage Coefficient

Very little literature exists on the estimation of the storage coefficient (R) for the Clark Unit Hydrograph. Clark (1945) had originally proposed a relation between T_c and R since they can both be defined by locating the inflection point of a runoff hydrograph (refer to Figure 5 2). The Corps of Engineers discuss the development of regionalized relations for T_c and R as functions of watersheds characteristics in Training Document No. 15 (U.S. Army Corps of Engineers, 1982b). According to Corps procedures, T_c and R are estimated from relations of $T_c + R$ and $R / (T_c + R)$ as functions of watershed characteristics. These forms of empirical equations indicate an interrelation of T_c and R, and such dependence was observed in the database, as discussed in the Documentation Manual. The equation for estimating R for Maricopa County is:

$$R = 0.37\, T_c^{\,1.11} A^{\,-0.57} L^{\,0.80} \tag{5.8}$$

where:

R = storage coefficient, in hours,

T_c = time of concentration, in hours,

A = drainage area, in square miles, and

L = length of flow path, in miles.

5.5.3 Time-Area Relation

Either a synthetic time-area relation must be adopted or the time-area relation for the watershed must be developed. If a synthetic time-area relation is not used, the time-area relation is developed by dividing the watershed into incremental runoff producing areas that have equal incremental travel times to the outflow location. This is a difficult task and well defined and reliable procedures for this are not available. The following general procedure is often used:

1. Use a topographic map of the watershed to trace along the flow path the distance from the hydraulically most distant point in the watershed to the outflow location; this defines L in both Equations 5.5 and 5.8.

2. Draw isochrones on the map to represent equal travel times to the outflow location. These isochrones can be established by considering the land surface slope and resistance to flow, and also whether the runoff would be sheet flow or would be concentrated in watercourses. A good deal of judgement and interpretation is required for this.

3. Measure and tabulate the incremental areas (in an upstream sequence) as well as the corresponding travel time for each area.

4. Prepare a graph of travel time versus contributing area (or a dimensionless graph of time as a percent of T_c versus contributing area as a percent of total area). The dimensionless graph is preferred because this facilitates the rapid development of new time-area relations should there be a need to revise the estimate of T_c.

Synthetic time-area relations can be used such as the default relation in the HEC-1 program:

$$A^* = 1.414\,(T^*)^{1.5} \qquad \textit{for } 0 \le T^* \le 0.5 \tag{5.9}$$

$$1 - A^* = 1.414\,(1 - T^*)^{1.5} \qquad \textit{for } 0.5 \le T^* \le 1.0$$

where:

A^* = contributing area in percent of total area and

T^* = time in percent of T_c.

Equation 5.9 is a symmetric relation and is not recommended for most watersheds in Maricopa County.

Two other dimensionless time-area relations have been developed during the reconstitution of recorded rainfall-runoff events as described in the Documentation Manual. These dimensionless relations for urban and natural watersheds are shown in Figures 5.6 and 5.7. Each of those figures show a synthetic time-area relation and shaded zone where the time-area relation is expected to lie. For an urban watershed, the synthetic time-area relation of Figure 5.6 is recommended, and for a natural (undeveloped) watershed the synthetic time-area relation of Figure 5.7 is recommended. If a time-area relation is developed from the watershed map, which is generally recommended for unusually shaped watersheds, then the resulting relation should lie within the shaded zones in either Figure 5.6 or 5.7. The HEC-1 default time-area relation is shown for comparison in each figure. Tabulated values of the dimensionless time-area relations are shown in Table 5.4.

FIGURE 5.6
SYNTHETIC TIME-AREA RELATION FOR URBAN WATERSHED

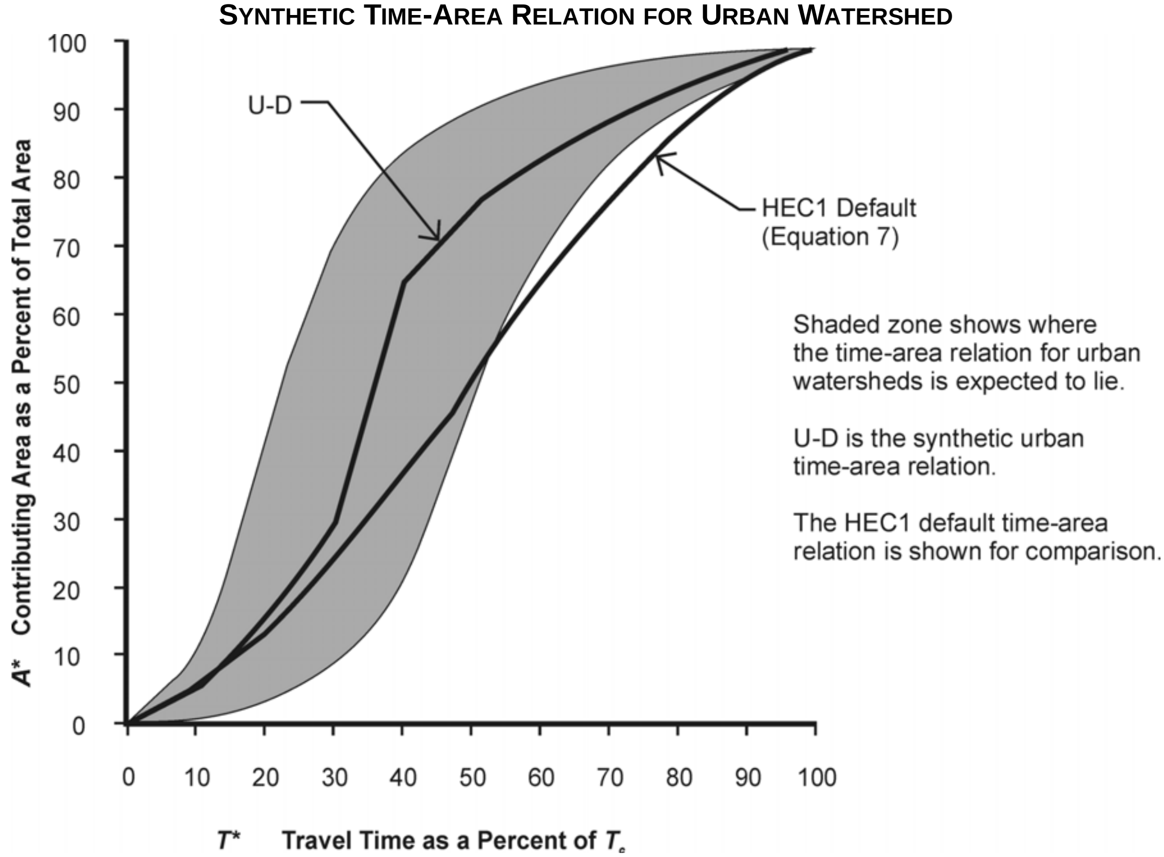


FIGURE 5.7
SYNTHETIC TIME-AREA RELATION FOR NATURAL WATERSHEDS

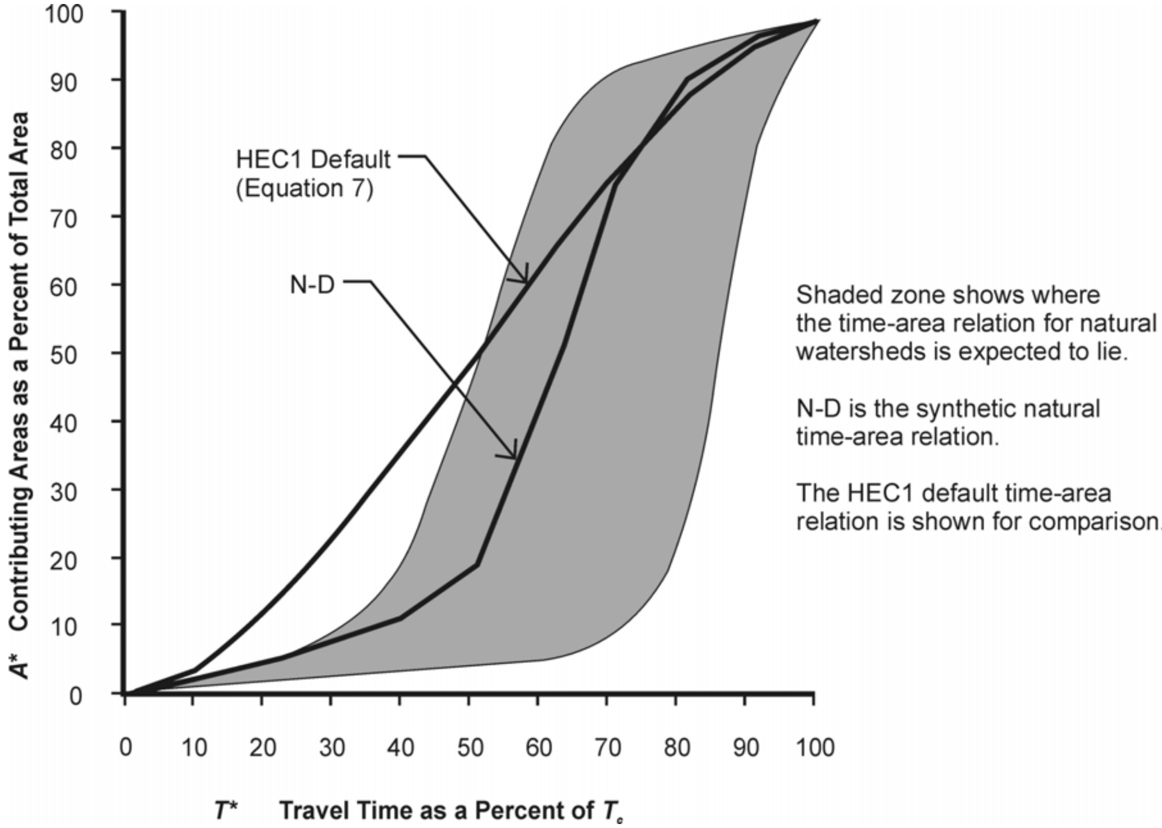


TABLE 5.4
VALUES OF THE SYNTHETIC DIMENSIONLESS TIME-AREA RELATIONS FOR THE CLARK UNIT HYDROGRAPH

Time, as a percent of Time of Concentration	Contributing Area, as a Percent of Total Area		
	Urban Watersheds	Natural Watersheds	HEC-1 Default
(1)	(2)	(3)	(4)
0	0	0	0.0
10	5	3	4.5
20	16	5	12.6
30	30	8	23.2
40	65	12	35.8
50	77	20	50.0
60	84	43	64.2
70	90	75	76.8
80	94	90	87.4
90	97	96	95.5
100	100	100	100.0

5.6 S-GRAPHS

An S-graph is a dimensionless form of a unit hydrograph and it can be used in the place of a unit hydrograph in performing flood hydrology studies. The concept of the S-graph dates back to the development of the unit hydrograph itself, although the application of S-graphs has not been as widely practiced as that of the unit hydrograph. The use of S-graphs has been practiced mainly by the U.S. Army Corps of Engineers, Los Angeles District, and the U.S. Bureau of Reclamation (USBR).

An example of an S-graph from Design of Small Dams (USBR, 1987) is shown in Figure 5.8. The discharge scale is expressed as percent of ultimate discharge (Q_{ult}), and the time scale is expressed as percent lag. Lag is defined as the elapsed time, usually in hours, from the beginning of an assumed continuous series of unit rainfall excess increments over the entire watershed to the instant when the rate of resulting runoff equals 50 percent of the ultimate discharge. The intensity of rainfall excess is 1 inch per duration of computation interval (Δt). An equivalent definition of lag is the time for 50 percent of the total volume of runoff of a unit hydrograph to occur. It is to be noted that there are numerous definitions for lag in hydrology and the S-graph lag should not be calculated by methods that are not consistent with this definition.

Ultimate discharge is the maximum discharge that would be achieved from a particular watershed when subjected to a continuous intensity of rainfall excess of 1 inch per duration (Δt) uniformly over the basin. Ultimate discharge (Q_{ult}), in cubic feet per second (cfs), can be calculated from Equation 5.10:

$$Q_{ult} = \frac{645.33A}{\Delta t} \tag{5.10}$$

where:

A = drainage area, in square miles, and

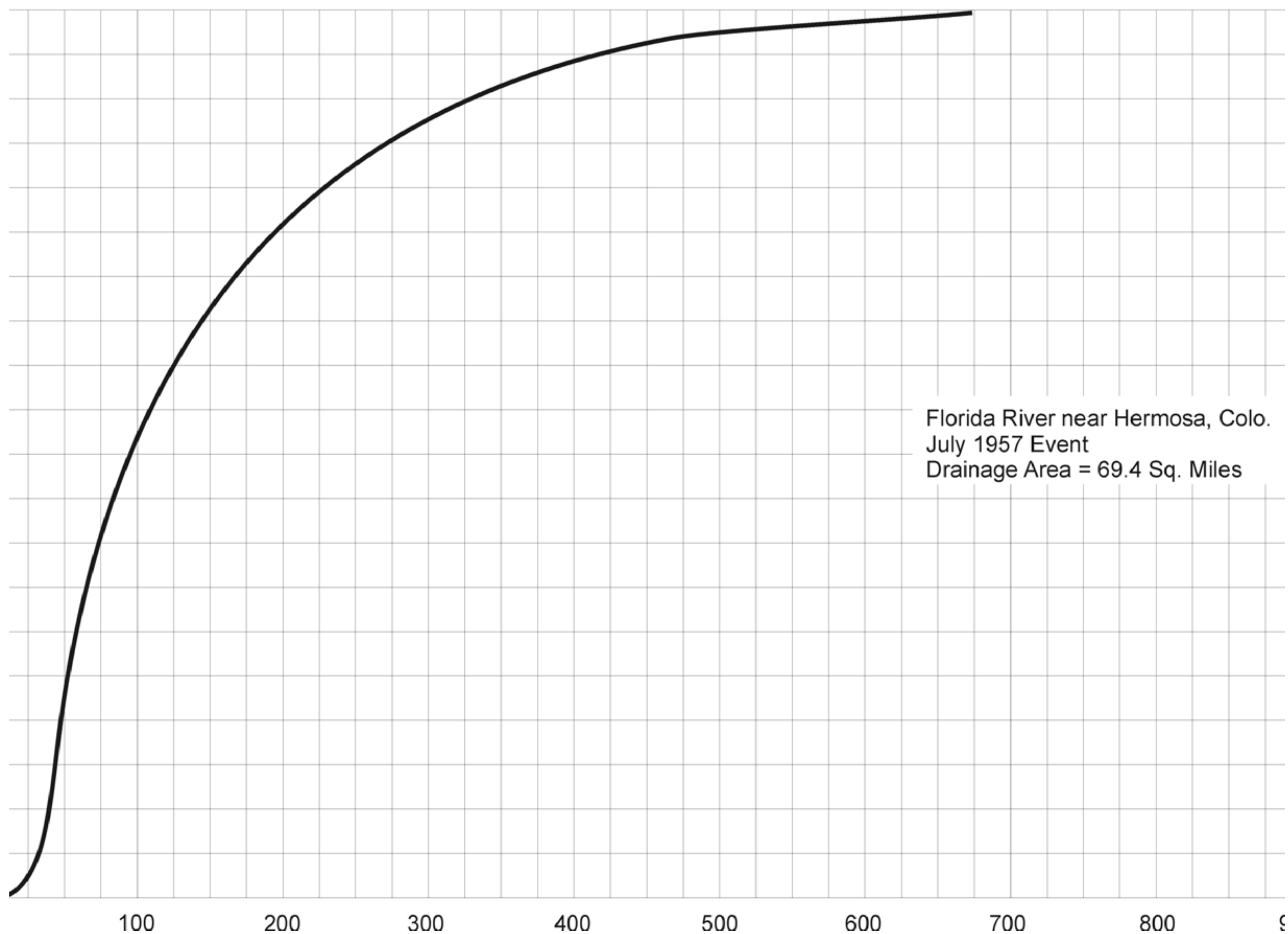
Δt = duration of the 1 inch of rainfall excess, in hours.

S-graphs are developed by summing a continuous series of unit hydrographs, each lagged behind the previous unit hydrograph by a time interval that is equal to the duration of rainfall excess for the unit hydrograph (Δt). The resulting summation is a graphical distribution that resembles an S-graph except that the discharge scale is accumulated discharge and the time scale is in units of measured time. This graph is terminated when the accumulated discharge equals Q_{ult} which occurs at a time equal to the base time of the unit hydrograph less one duration interval. The basin lag can be determined from this graph at the time at which the accumulated discharge equals 50 percent of Q_{ult}. This summation graph is then converted to a dimensionless S-graph by dividing the discharge scale by Q_{ult} and the time scale by lag.

In practice, S-graphs have generally been developed by reconstituting observed floods to define a representative unit hydrograph and then converting this to an S-graph. Prior to the advent of computerized models, such as HEC-1, flood reconstitution was a laborious task of rainfall and hydrograph separation along with numerous hand-cranked simulations to define the representative unit hydrograph. Modern S-graph development generally relies on use of optimization techniques, such as coded into HEC-1, to identify unit hydrograph parameters that best reproduce the observed flood.

Although an S-graph is completely dimensionless and does not have a duration of rainfall excess associated with it as does a unit hydrograph, its general shape and the magnitude of lag is influenced by the distribution of rainfall over the watershed and the time distribution of the rainfall. Therefore, the transposition of an S-graph from a gaged watershed to application in another watershed must be done with consideration of both the physiographic characteristics of the watersheds and the hydrologic characteristics of the rainfalls for the two watersheds.

FIGURE 5.8
EXAMPLE OF AN S-GRAPH FROM DESIGN OF SMALL DAMS (USBR, 1987)



5.6.1 Limitations and Applications

S-graphs are empirical, lumped parameters that represent runoff characteristics for the watershed for which the S-graph was developed. S-graphs that are developed from recorded runoff data from one watershed can be applied to another watershed only if the two watersheds are hydrologically and physiographically similar. In addition, a study for the Flood Control District of Maricopa County (Sabol, 1987) has demonstrated the shape of S-graphs is significantly affected by storm characteristics, particularly the maximum intensity of the rainfall. Therefore, it may not be advisable to adopt S-graphs that have been developed from one hydrologic zone and to apply those to watersheds in other hydrologic zones because of possible differences in rainfall characteristics in the two zones that may affect the shape of the S-graph. Application of S-graphs requires the selection of an appropriate S-graph and the estimation of the one parameter, basin lag. Four S-graphs have been selected for use in Maricopa County and a method to estimate lag is provided.

The USBR has revised the Flood Hydrology Studies chapter of Design of Small Dams (USBR, 1987), and it has identified S-graphs for application in six generalized regional and physiographic type of watersheds. The USBR has issued a Flood Hydrology Manual (Cudworth, 1989) that contains extensive discussion of flood hydrology in general, and S-graphs in particular. Both of these references should be consulted before using S-graphs. The S-graph has been adopted as the unit hydrograph procedure by Orange County and San Bernardino County, California, and selected S-graphs are presented in the hydrology manuals for those counties. The S-graphs in those hydrology manuals have been selected primarily from S-graphs that previously had been defined by the U.S. Army Corps of Engineer, Los Angeles District from a rather long and extensive history of analyses of floods in California.

An S-graph can, in theory, be used in any application for which an unit hydrograph can be used. In practice an S-graph must be first converted to an unit hydrograph, and this can be done by one of two methods. First, the S-graph can be converted to an unit-hydrograph manually; or second, the S-graph can be converted to an unit hydrograph by use of the DDMSW program. The DDMSW program outputs the HEC-1 input file with the S-graph converted to a unit hydrograph, and the unit hydrograph is written to a HEC-1 input file using the UI (given Unit Graph) record. The use of DDMSW greatly facilitates the use of S-graphs.

Although the S-graph is completely dimensionless and does not have a rainfall excess duration associated with it, the unit hydrograph does require the specification of the duration. In general, the same rules and recommendations apply to the S-graph as were made for the Clark Unit Hydrograph; that is, the duration (computation interval, NMIN) selected for the development of the unit hydrograph from a S-graph should equal about 0.15 times the lag. A duration (NMIN) in the range 0.10 to 0.25 times the lag is usually acceptable.

5.6.2 Sources of S-Graphs

S-graphs for Maricopa County have been selected from a compilation of S-graphs for the Southwestern United States (Sabol, 1987a) and an evaluation of S-graphs (Sabol, 1993a) used in the Unit Hydrograph Study (Sabol, 1987b). The sources of S-graphs for that compilation were reports and file data of the U.S. Army Corps of Engineers, Los Angeles District, and the USBR, as well as data collected for the Unit Hydrograph Study from gaged watersheds in Walnut Gulch, Tucson, Albuquerque, Denver, and Wyoming.

5.6.3 S-Graphs for Use in Maricopa County

The four S-graphs selected for use in flood hydrology studies in Maricopa County are the Phoenix Mountain, the Phoenix Valley, the Desert/Rangeland, and Agricultural S-graphs. The Phoenix Mountain S-graph is to be use in flood hydrology studies of watersheds that drain predominantly mountainous terrain, such as Agua Fria River above Rock Springs, New River above the Town of New River, the Verde River, Tonto Creek, and the Salt River above Phoenix. Although the Corps of Engineers developed a separate S-graph for Indian Bend Wash, it is nearly identical to the Phoenix Mountain S-graph, which may also be appropriate for Indian Bend Wash.

The Phoenix Valley S-graph is appropriate for flood hydrology studies of watersheds that have little topographic relief and/or urbanized watersheds. However, the Clark method is still the preferred unit hydrograph method for use in urban areas in Maricopa County. The Desert/ Rangeland S-graph is appropriate for use in natural areas with little to moderate relief, such as foothills, distributary flow areas, and other undeveloped desert areas. The Agricultural S-graph as the name suggests should be used for areas under agricultural crops like cotton, wheat, or vegetables. Table 5.6 summarizes the four S-graphs and describes their general areas of applicability.

The four S-graphs are shown in Figure 5.9 and the coordinates of the graphs listed in Table 5.5. The selection of S-graph should be made based on a comparison of the watershed of interest to the watershed(s) used to develop the various S-graphs.

FIGURE 5.9
S-GRAPHS FOR USE IN MARICOPA COUNTY

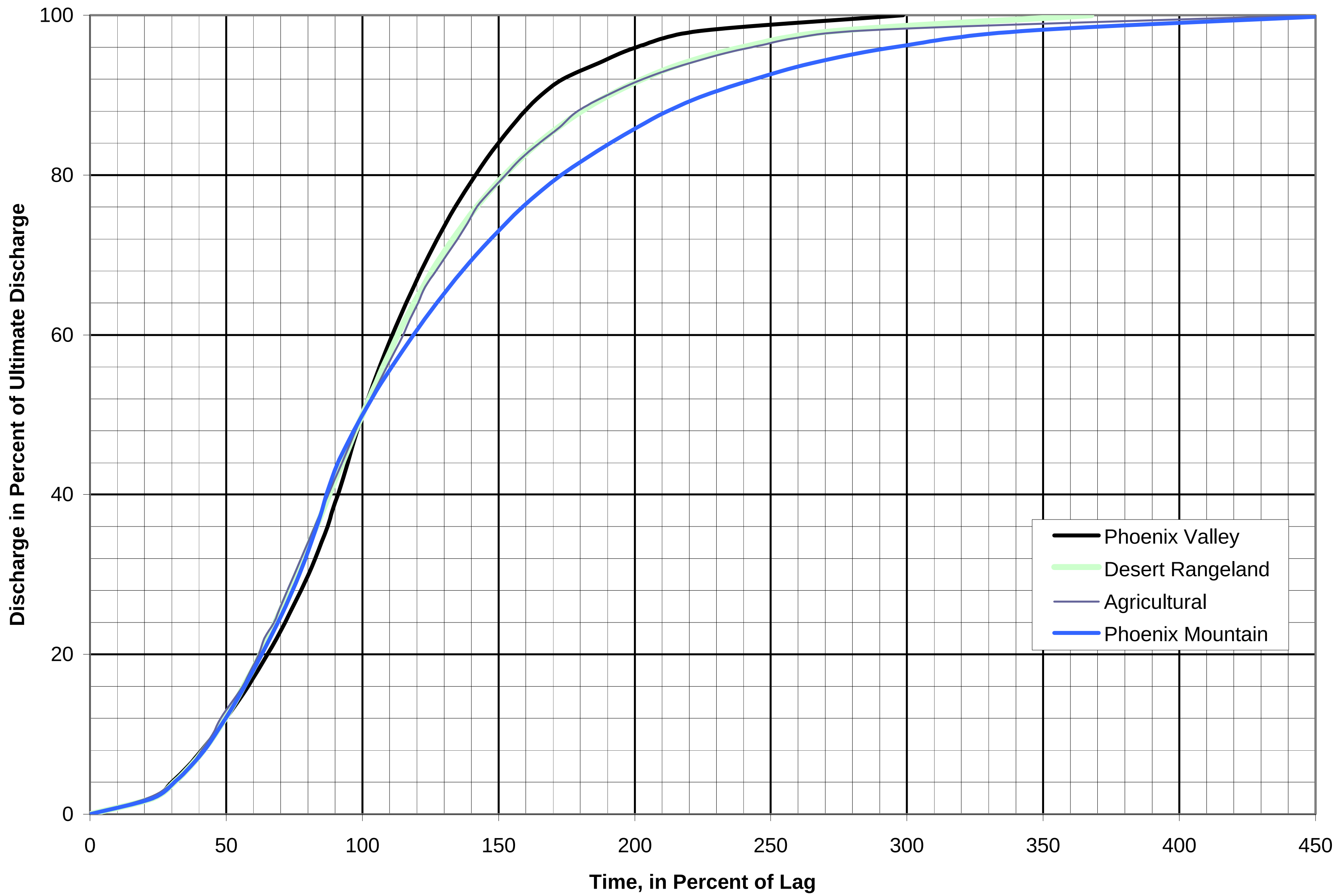


TABLE 5.5
TABULATION OF COORDINATES FOR S-GRAPHS

Percent Ultimate Discharge	Time in Percent Lag			
	Phoenix Valley	Phoenix Mountain	Desert/Rangeland	Agricultural
0	0.0	0.0	0.0	0.0
2	23.0	23.0	23.0	21.0
4	30.0	31.0	31.0	31.0
6	36.0	37.0	36.9	37.0
8	41.0	42.0	41.7	41.0
10	45.7	46.0	45.9	45.0
12	50.0	49.8	49.7	48.0
14	54.1	53.4	53.2	52.0
16	58.0	56.8	56.4	56.0
18	61.7	60.0	59.7	59.0
20	65.2	63.1	62.5	62.0
22	68.5	66.1	65.3	64.0
24	71.6	69.0	68.0	67.5
26	74.6	71.8	70.6	70.0
28	77.5	74.4	73.2	72.5
30	80.2	76.8	75.7	75.0
32	82.7	79.1	78.3	77.5
34	85.0	81.2	80.7	80.0
36	87.2	83.2	83.1	82.5
38	89.0	85.1	85.5	85.0
40	91.1	86.8	87.9	87.5
42	92.9	88.8	90.3	90.0
44	94.6	91.0	92.7	92.5
46	96.3	93.8	95.1	95.0
48	98.1	96.8	97.5	97.5
50	100.0	100.0	100.0	100.0
52	102.0	103.4	102.5	103.0
54	104.1	107.0	105.1	106.0
56	106.3	110.8	107.6	109.0
58	108.6	114.7	110.3	112.0
60	111.0	118.7	113.0	115.0
62	113.5	122.9	115.9	117.5
64	116.1	127.3	119.0	120.5
66	118.8	131.9	122.3	123.0
68	121.6	136.7	125.6	127.0
70	124.5	141.7	129.3	131.0
72	127.5	147.1	133.2	135.0
74	130.7	152.8	137.4	138.6
76	134.1	158.8	141.9	142.0
78	137.7	165.5	146.8	147.0
80	141.5	172.9	152.1	152.5
82	145.5	172.9	152.1	158.0
84	149.9	191.0	164.5	165.0
86	154.6	201.0	172.0	172.5
88	159.6	212.0	180.4	179.0
90	165.6	226.0	190.7	190.0
92	173.6	244.0	202.9	203.0
94	186.6	265.0	217.9	220.0
96	200.6	295.0	239.6	243.0
98	223.6	342.0	273.2	280.0
100	298.6	462.0	367.7	448.0

5.6.4 Estimation of Lag

The application of an S-graph requires the estimation of the parameter, basin lag. A general relationship for basin lag as a function of watershed characteristics is given by Equation 5.11:

$$Lag = C\left(\frac{LL_{ca}}{S^p}\right)^m \qquad (5.11)$$

where:

Lag = basin lag, in hours,

L = length of the longest watercourse, in miles,

L_{ca} = length along the watercourse to a point opposite the centroid, in miles,

S = watercourse slope, in feet per mile,

C = coefficient, and

m and p = exponents.

The Corps of Engineers often uses $C = 24K_n$, where K_n is the estimated mean Manning's n for all the channels within an area, and $m = 0.38$. The USBR (1987) has recommended that $C = 26K_n$ and $m = 0.33$. Both sets of values in Equation 5.11 will often result in similar estimates for Lag. Traditionally the exponent, p, on the slope is equal to 0.5.

It should be noted that K_n is a measure of the hydraulic efficiency of the watershed and it is not necessarily a constant for a given watershed for all rainfall depths and rainfall intensities. As rainfall depth and/or rainfall intensity increases the efficiency of runoff increases and K_n decreases. Therefore, some adjustment in K_n should be made for use with rainfalls of different magnitudes (frequencies). Generally, K_n is the smallest for extreme floods such as PMFs and increases as the frequency of event increases.

5.6.4.1 Selection of K_n

The selection of a representative K_n value for a particular watershed is an inherently subjective process. However, some guidelines are given for the selection of K_n in Maricopa County in conjunction with the four recommended S-graphs. Table 5.6 contains a summary of these guidelines. Additional guidance may be gleaned from the calculated K_n values for numerous watersheds provided in Appendix D, Section 2. Care should be taken to keep in mind the limitations discussed above when selecting K_n for any given watershed.

Several graphical relations are available for estimating basin lag. One such relation (U.S. Army Corps of Engineers, 1982a) is shown in Appendix D, Section 2. Several other relations that should be consulted when using S-graphs are contained in Design of Small Dams (USBR, 1987) and the USBR Flood Hydrology Manual (Cudworth, 1989).

TABLE 5.6
S-GRAPHS AND K_n VALUES

S-Graph Type	Description	K_n Min	K_n Avg	K_n Max	Description
Phoenix Valley	Very shallow slopes and/or partially urbanized	0.015	---	0.15	variations dependent upon slope, degree of urbanization and connected impervious areas and development of organized drainage improvements; extreme high values may be appropriate in very flat areas with little or no drainage network
Phoenix Mountain	Mountain	0.045	0.05	0.055	quite rugged, with sharp ridges and narrow, steep canyons through which watercourses meander around sharp bends, over large boulders, and considerable debris obstruction; ground cover, excluding small areas of rock outcrops, includes many trees and considerable underbrush; no drainage improvements
	Foothills	0.027	0.03	0.033	gently rolling, with rounded ridges and moderate side slopes; watercourses meander in fairly straight channels with some boulders and lodged debris; ground cover includes scattered brush, cactus and grasses; no drainage improvements
Desert/Rangeland	Gently sloping natural areas including distributary flow areas	0.020	0.025	0.03	variations from minimum to maximum roughness due to degree of definition of watercourses, extent of vegetation, and land surface hydraulic condition
Agricultural	Actively cultivated areas with crops	0.06	0.10	0.15	variations from minimum to maximum dependent upon slope, crop type and density

Note: The majority of K_n data upon which these values are based come from rainfall runoff events of magnitude less than the 100-year event. Therefore, selected K_n values for a given design storm need to be evaluated for the purposes of modeling a particular watershed response to that design storm.

5.7 PROCEDURES

Procedures for calculating the unit hydrograph parameters are provided in the following sections. Notes and general guidance on the application of these procedures and the methodologies presented in this chapter are provided along with a detailed example in Chapter 9, Section 9.4.

5.7.1 Clark Unit Hydrograph

1. From an appropriate map of the watershed, measure drainage area (A) and the values of L and S.
2. If S is greater than 200 ft/mi, adjust the slope using Table 5.2 or Figure 5.4.
3. Using either Figure 5.5 or Table 5.3, select a resistance coefficient (K_b) for the basin or subbasin based on a resistance classification and the drainage area (in acres). For a basin or subbasin of mixed classification;
    - A representative K_b can be interpolated from Figure 5.5, or
    - An arithmetically averaged K_b can be calculated based on the area of each unique K_b present in the basin or subbasin.
4. Calculate T_c as a function of i using Equation 5.5
5. Enter the following data into an HEC-1 input file:
    - Design rainfall per the methodology and procedures in Chapter 2;
    - Basin area;
    - Rainfall loss data per the methodologies and procedures in Chapter 4; and
    - Clark unit hydrograph parameters (values set to zero).
6. Run HEC-1 with the input file from Step 5 at an output level of zero for each subbasin. Using the T_c worksheet (Appendix D, Section 1), tabulate the period of peak rainfall excess for each subbasin and compute the average intensities to a time greater than the expected T_c.
7. Construct the graph of average rainfall excess intensity vs. time and calculate T_c by iteration.
8. Calculate R using Equation 5.6.
9. Select the appropriate time-area relation for the basin or subbasin.

As an alternative to the above procedures, the DDMSW program will compute the rainfall excess directly and perform the necessary iterations to compute the T_c and R parameters.

5.7.2 S-Graph

1. From an appropriate map of the watershed, measure drainage area (A), L, L_{ca} and S.
2. Calculate the basin factor $\frac{LL_{ca}}{S^{0.5}}$.
3. Using the data in Appendix D, Section 2 or the tables in the Design of Small Dams or the USBR Flood Hydrology Manual, attempt to identify watersheds of the same physiographic type and similar drainage area and basin factor. Make a list of the watersheds with similar drainage areas and basin factors and tabulate the estimated value of K_n for those watersheds and the measured lag.
4. Estimate K_n for the watershed by inspection of the tabulation from Step 3.
5. Calculate the coefficient (C) and select the value of the exponent (m) corresponding to the source (Corps of Engineers or USBR) that was used to estimate K_n. If the source of K_n is unknown, then use the Corps of Engineers version of Equation 5.11.
6. Using Equation 5.11, calculate the basin lag. Compare this value to the measured lags of watersheds from Step 3.
7. Select an appropriate computational time interval (NMIN) and compute Q_{ult} using Equation 5.10.
8. Select an appropriate S-Graph and tabulate the percent Q_{ult}, percent lag and the accumulated time.
9. Transform the S-Graph into an X-duration (NMIN) unit hydrograph using linear interpolation with Δt = NMIN.
10. Adjust the “tail” region of the S-Graph by lagging that portion by Δt and subtracting the ordinates.

As an alternative to the above procedure, the DDMSW will transform the S-Graph to a unit graph automatically.

6 MULTIPLE FREQUENCY MODELING

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6.1 BACKGROUND

Originally, the *Hydrology Manual* was intended to be used for the development of flood discharges and runoff volumes resulting from infrequent storms, such as the 100-year rainfall. Data that were collected and used in the selection and development of the methods, techniques and parameters are representative of infrequent storms. While it was recognized that the application of the methods, techniques and procedures may not be appropriate for more frequent storms, this limitation was not perceived as a significant issue at that time.

Recently, there has been an increasing need for runoff magnitudes from more frequent storms, particularly in regard to the design of storm drains, but also for regulatory and planning purposes. However, use of the methods, techniques and parameters presented in the preceding chapters may result in the overestimation of runoff magnitudes for those types of events. The threshold at which this occurs often is the 10-year recurrence interval. Several different alternative approaches were considered that could be used in place of or to supplement the methods, techniques and parameters presented in the preceding chapters. Each alternative method was evaluated in regard to the three benchmarks (accuracy, practicality and reproducibility) that were used to evaluate the original methods, techniques and parameters. The alternative approach to be used in Maricopa County for the estimation of runoff for more frequent storms is a ratio that is applied to the 100-year runoff hydrographs.

6.2 APPROACH

Ratios for the 2-, 5- and 10-year recurrence intervals are based on analysis of USGS gage data for watersheds throughout the State of Arizona. That data reflects the wide range of hydrologic and physiographic characteristics that exist in Arizona. This variability was considered in the analysis in regard to the conditions that are specific to Maricopa County.

For reasons of practicality and to facilitate reproducibility, a single ratio for the 2-, 5- and 10-year recurrence intervals is provided that represents average conditions in Maricopa County. These values are listed in Table 6.1 and can be used for both local and general storms for drainage areas of any size, degree of development or other hydrologic and physiographic conditions.

TABLE 6.1
RATIOS TO 100-YEAR FLOOD HYDROGRAPHS FOR THE 2-, 5- AND 10-YEAR RECURRENCE INTERVAL FLOODS

Recurrence Interval	Ratio %
2	10
5	25
10	35

This approach should be used when the results for the 2-, 5- and 10-year flood (peaks and volumes) using the methods, techniques and parameters in the preceding chapters are unreasonable. The reasonableness “test” applies to model results (peak discharges and runoff volumes) as well as to the HEC-1 input parameters, particularly for the unit hydrograph. This alternative method using the ratios from Table 6.1 does not preclude the use of another method or the use of different (site specific) ratios with prior approval from the Flood Control District, or local jurisdiction.

6.3 IMPLEMENTATION IN HEC-1

The ratio for the desired recurrence interval is coded into the 100-year HEC-1 model on field 3 of the subbasin area (BA) record for each subbasin. Alternatively, for a single storm analysis the ratio(s) can be coded into the 100-year HEC-1 model on the multiratio (JR) record. In addition to coding the ratio(s) on this record, the IRTIO variable in field 1 must be set to FLOW to ratio the runoff not the precipitation. The JR record cannot be used for a multiple storm analysis due to a conflict with the JD record used to define the index areas.

7 CHANNEL ROUTING

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7.1 GENERAL

Channel routing involves generation of an outflow hydrograph for a reach where an inflow hydrograph is specified. A reach is either an open channel with certain geometrical/structural specifications, or a pipe with open channel flow. This type of application assumes that the flow is not confined, and that surface configuration, flow pattern and pressure distribution within the flow depend on gravity. It also assumes that there is no movement of the bed or banks. In addition no backwater effects are considered.

A routing technique is normally required for a multi-basin design where flow is to be moved through time and space from one flow concentration point to the next. For the purposes of this manual, two types of open channels, natural and urbanized, are considered. The preferred method for most applications in Maricopa County is Normal-Depth routing. Normal-Depth routing can be used for both natural and artificial channels in both urbanized and non-urbanized watersheds. Kinematic Wave routing may be used in urbanized watersheds and for natural channels where reductions in peak discharge due to attenuation is not anticipated. The Kinematic Wave method is limited to simple prismatic channel geometrics that include non-pressurized closed conduits. Muskingum routing may be used for large natural channels where parameter calibration data exists. The Muskingum-Cunge routing may be used for both natural and artificial channels.

Notes and general guidance on the parameter development and application of each of these methods are provided along with a detailed example in Chapter 9, Section 9.5.

7.2 NORMAL-DEPTH ROUTING

The Normal-Depth routing method uses the Modified Puls routing method with storage and outflow data being computed by HEC-1 from channel characteristics entered by the user into the HEC-1 data file. This method is physically based that simulates attenuation due to overbank storage.

7.2.1 Parameter Selection

Input data for Normal-Depth routing include the estimation of a representative eight-point cross section, the energy slope (or bed slope), reach length and Manning's n values for both the main channel and overbanks. In addition to those physical parameters, this method also requires the input of the number of routing steps (NSTPS) to be used in the computations. This is a calibration parameter that is directly related to the degree of attenuation introduced in the computations. This parameter is also a function of the model computational time interval, NMIN, as given by the following.

$$\mathrm{NSTPS} = \frac{((L)/V_{avg})}{\mathrm{NMIN}} \tag{7.1}$$

where:

$NSTPS$	=	number of routing steps, a dimensionless integer.
L	=	reach length, in feet.
V_{avg}	=	velocity of flood wave, in ft per minute.
$NMIN$	=	hydrograph computation time interval, in minutes.

For a complete description of the use and application of Normal-Depth routing refer to the HEC-1 User's Manual. A second applicable reference is Hoggan (1989). Refer to section 9.5 for guidance in calibration of NSTPS.

7.3 KINEMATIC WAVE ROUTING

The Kinematic Wave routing as described in HEC-1 can be applied for routing of overland flow, collector channels and the main channel. However, for the purposes of this manual, the overland flow option of the Kinematic Wave will not be used.

7.3.1 Collector Channel

Modeling of flow at a point where it becomes channel flow to a point where it enters the main channel is done as a collector channel element. It is assumed that the flow along the path of the channel is uniformly distributed. This is a proper assumption for a case when overland flow runs directly into a gutter. It is also a reasonable approximation of the flow as it passes through a storm drain system from a catch basin and the collector pipes along the collector channels.

7.3.2 Main Channel

The main channel element can be used to route inflow from an upstream subbasin or a combination of inflows from collector channels along a subbasin. The flow is assumed to be uniformly distributed, which appears to be a reasonable assumption when the flow is received from collector channels at several locations.

7.3.3 Parameter Selection

The data requirement for Kinematic Wave channel routing include surface drainage area, channel length and slope, channel shape and geometry, Manning's n, and the inflow hydrograph. The designer is referred to the HEC-1 manual for the proper selection of these parameters.

When working with the Kinematic Wave method, it is important to be familiar with the computational procedures inherent in the model. In order to solve the governing equations, which theoretically describe the Kinematic Wave method, proper selection of time step and reach length are required. The designer will specify a channel reach length and a computational time step for the inflow hydrograph. This time step could very well be different from the one selected by the computer for computational purposes. Furthermore, the computer will use this information to select distance intervals based on the given reach length.

The computational process could unrealistically attenuate the outflow peak. It appears that a longer reach length results in more attenuation. To overcome this problem, more recent versions of HEC-1 will calculate the outflow peak by applying both the time step selected by the designer as well as the one selected by the program. If the resulting peaks are not reasonably close, the designer can modify the selected time step or the reach length to improve the calculations. It should be noted that the program will compare peak flow values for the main channel and not the collector channels.

7.4 MUSKINGUM ROUTING

Flow routing through natural channels can be accomplished by applying the Muskingum Routing technique. The main characteristic of natural channels with respect to routing is that the outflow peak can be drastically attenuated through storage loss, a process which is simulated by Muskingum routing.

7.4.1 Parameter Selection

Application of Muskingum routing requires input values for parameters X and K. Parameter X has a range of values from 0.0 to 0.5, where 0.0 represents routing through a linear reservoir and 0.5 indicates pure translation. Parameter K indicates the travel time of a floodwave through the entire routed reach. There are several methods which can be used to estimate K such as average flow velocity adjusted by a celerity factor, the time difference between peak inflow and peak outflow, or by using stage-discharge relationships. For more details the reader is referred to the HEC-1 manual and Section 6.6 of this manual. Once again, since the computational method within HEC-1 may result in an unstable solution, parameters K, X and NSTPS (number of steps) must be checked to insure that an adequate number of subreaches is used.

In those rare situations that observed inflow and outflow hydrographs are available, K, X and NSTPS can be calibrated by trial and error to enable simulation of known outflow hydrographs. Chapter 5 of the USBR's Flood Hydrology Manual (Cudworth, 1989) is an excellent source of Muskingum routing information.

7.5 MUSKINGUM-CUNGE ROUTING

The Muskingum-Cunge routing method is based on the principle of hydraulic diffusivity, which simulates an attenuation of the flood peak through the routing reach. This method can be used for both man-made and natural channels where overbank flow is expected, provided the conveyance can be accurately described with an eight-point cross section. A complete description of Muskingum-Cunge applications and guidelines for parameter selection can be found in the September 1990, and later versions of the HEC-1 Flood Hydrograph Package, User's Manual.

7.5.1 Parameter Selection

Input data for Muskingum-Cunge routing include energy slope (or bed slope), reach length, and either the channel shape and a single Manning's n for a man-made channel, or an eight-point cross section with channel and overbank roughness coefficients for a natural channel.

8 INDIRECT METHODS

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8.1 GENERAL

The estimation of peak discharges by analytic methods (the Rational Method or by rainfall-runoff modeling (HEC-1 program)) is based on various assumptions, and in the case of HEC-1 modeling, requires the correct input of numerous parameters. Therefore, the resulting peak discharges that are computed by analytic methods should always be verified, to the extent possible, to guard against erroneous design discharges that can result from questionable assumptions and/or faulty model input.

Since the majority of discharge estimates are made for ungaged watersheds, usually only indirect methods can be used to check the discharge estimates obtained from either the Rational Method or rainfall-runoff modeling. When the watershed is gaged, or is near a gaging station, a flood frequency analysis can be performed and the results of that analysis can be used for design or used to check the results from analytic methods. The results of flood frequency analyses, because of variability of flooding in both the time and space regime, and because of uncertainties in the data and the analytic procedures, should also be checked by indirect methods.

True verification of design discharges cannot be made by any of the methods (analytic methods, flood frequency analyses, or indirect methods) because for none of these methods is there “absolute assurance” that the discharges obtained are the “true” representations of the flood discharge for a given frequency of flooding. However, the results of the various methods, when compared against each other and when qualitatively evaluated, can provide a basis for either acceptance or rejection of specific estimates of design discharges for watersheds in Maricopa County.

In this chapter, three indirect methods are presented for “verifying” flood discharges that are obtained by analytic methods.

Those procedures are:

1. A graph of seven unit peak discharge versus drainage area curves,

2. Graphs of estimated 100-year discharges versus drainage area for gaged watersheds in Arizona, and

3. Regression equations and data graphs for flood regions in Maricopa County.

In general, all three procedures should be used when verifying the results of analytic methods.

8.2 INDIRECT METHOD NO. 1 - UNIT PEAK DISCHARGE CURVES

Figure 8.1 presents 7 unit peak discharge relations and envelope curves. A brief description of each of those curves follows:

A. An envelope curve, based on a compilation of unusual flood discharges in the United States and abroad (data prior to 1941), by Craeger and others (1945).

B. An envelope curve of extreme floods in Arizona and the Rocky Mountain region developed by Matthai and published by Roeske (1978).

C. An envelope curve of peak streamflow data developed for Arizona by Malvick (1980).

D. An envelope curve of peak streamflow data for the Little Colorado River basin in Northern Arizona developed by Crippen (1982).

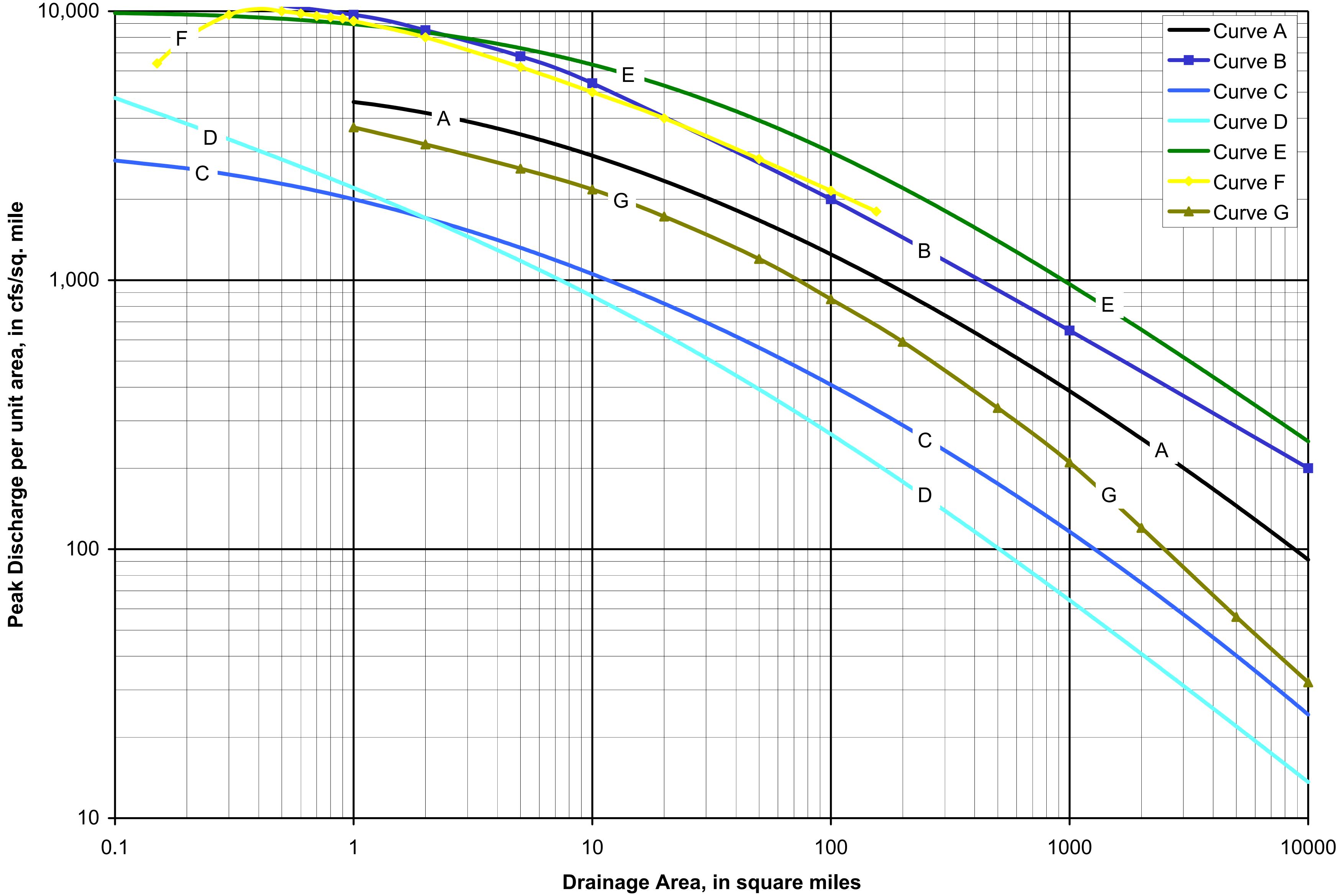
E. An envelope curve of peak streamflow data for Central and Southern Arizona developed by Crippen (1982).

F. An envelope curve of the largest floods in the semi-arid Western United States developed by Costa (1987).

G. An envelope curve of peak discharges for Arizona, Nevada and New Mexico developed by the U.S. Army Corps of Engineers (1988).

When using Figure 8.1, it must be noted that the curves represent envelopes of maximum observed flood discharges for different hydrologic regions.

FIGURE 8.1
PEAK DISCHARGE RELATIONS AND ENVELOPE CURVES



8.3 INDIRECT METHOD NO. 2 - USGS DATA FOR ARIZONA

The U.S. Geological Survey (USGS) provides streamflow and statistical data for 138 continuous-record streamflow-gaging stations and 176 partial-record gaging stations in Arizona (Garrett and Gellenbeck, 1991). The streamflow data were analyzed by the USGS by Log-Pearson Type 3 (LP3) analyses and flood magnitude-frequency statistics are provided in that report along with the maximum recorded discharge for each of the stations. Figure 8.2 is a plot of the 100-year peak discharge (from LP3 analyses) versus drainage area (for stations with drainage areas smaller than 2,000 square miles). Lines were fit to the data by least-squares of the log-transformed data. The equation for the 100-year peak discharge (Q_{100}) line is:

$$Q_{100} = 850\, A^{.54} \tag{8.1}$$

where:

A is the drainage area in square miles.

Figure 8.2 also shows 75 percent tolerance limit lines about the 100-year discharge line (Equation 8.1). The tolerance limits are a statistical measure of the spread of the data about that line.

As an aid to using Figure 8.2, that figure is reproduced with larger drainage area scales in Figures 8.3 and 8.4. Those larger scale plots of the data also show 75 percent tolerance limit lines about the 100-year discharge line (Equation 8.1).

A listing of the data that was used to produce Figures 8.2 through 8.4 is shown in Table 8.1. This table includes USGS streamflow-gaging station numbers, the associated drainage areas and the 100-year flood peak discharge estimates by LP3. Watershed characteristics for each of these gaging stations is provided in the USGS report (Garrett and Gellenbeck, 1991). A map of Arizona showing the locations of the gaging stations for this data compilation are shown in Figure 8.5.

FIGURE 8.2

100-YEAR PEAK DISCHARGE BY LP3 ANALYSIS

Source: 1989 USGS Basin Characteristic Report, Figure Adapted from the ADOT Hydrology Manual

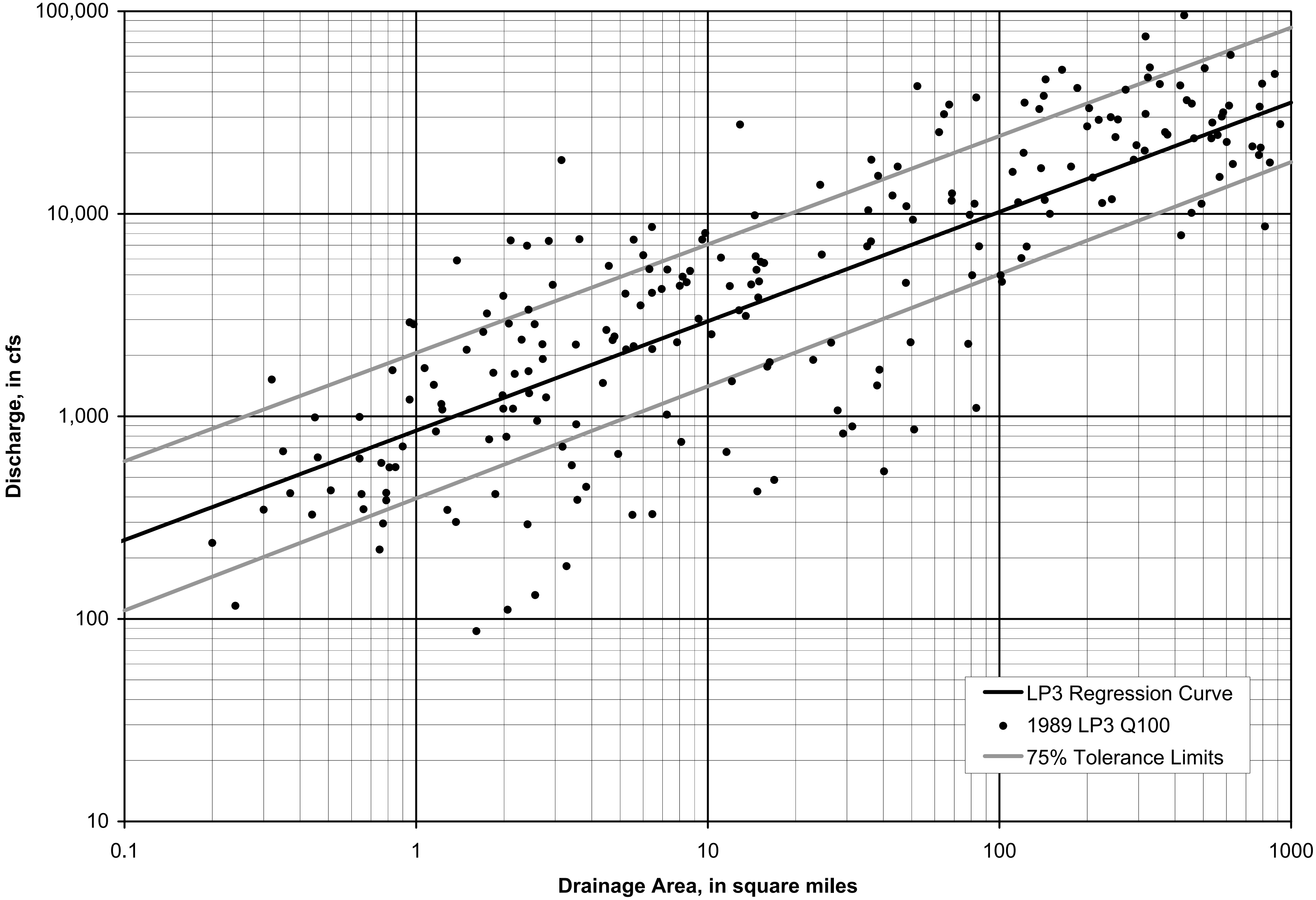


FIGURE 8.3
100-YEAR PEAK DISCHARGE BY LP3 ANALYSIS

Source: 1989 USGS Basin Characteristic Report, Figure Adapted from the ADOT Hydrology Manual

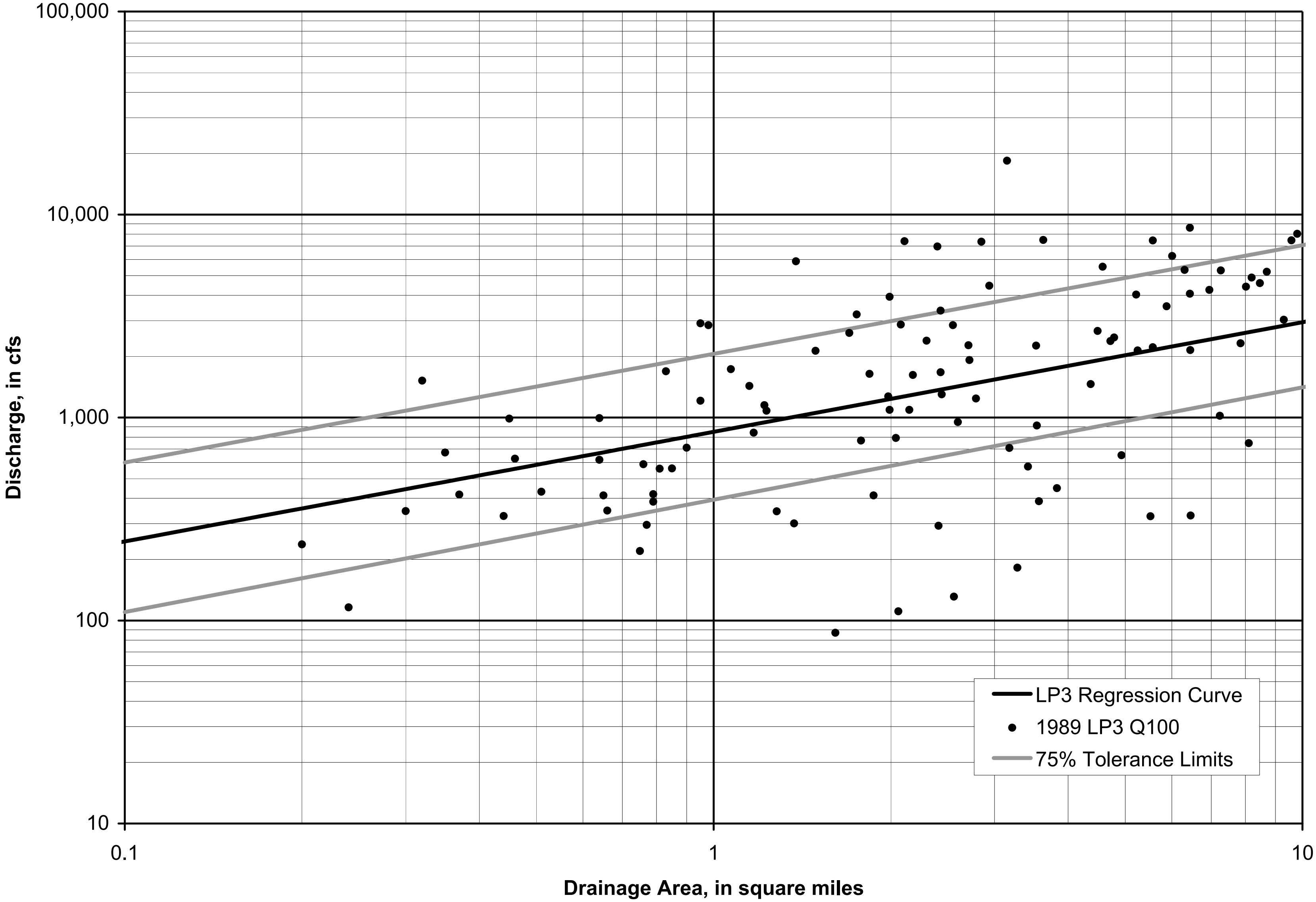


FIGURE 8.4
100-YEAR PEAK DISCHARGE BY LP3 ANALYSIS
Source: 1989 USGS Basin Characteristic Report, Figure Adapted from the ADOT Hydrology Manual

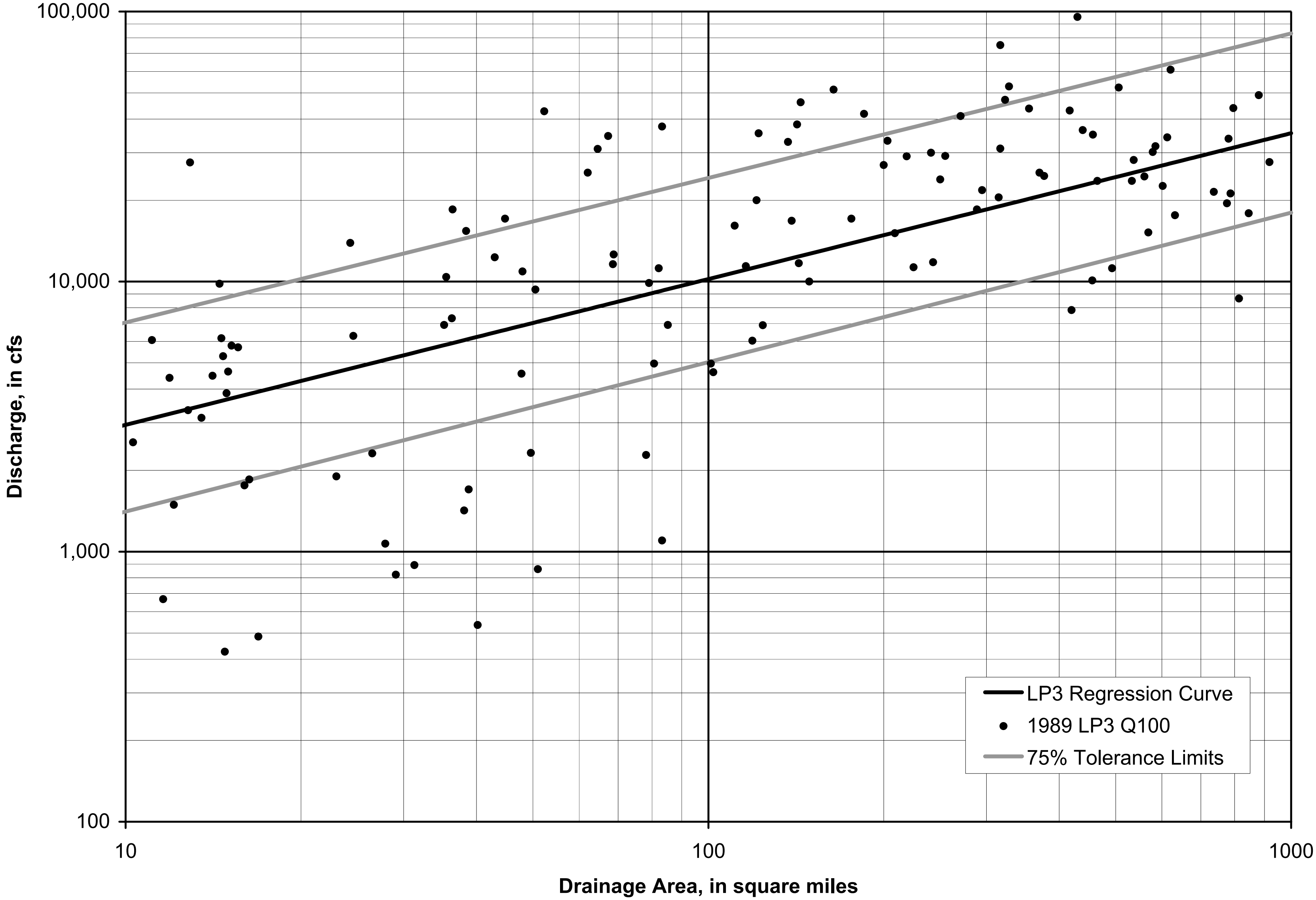


TABLE 8.1
USGS DATA LISTING FOR WATERSHEDS WITH DRAINAGE AREAS BETWEEN .1 AND 2,000 SQUARE MILES
(ordered by increasing drainage area)

Drainage Area	Gage No.	LP3 Q_{100}	Drainage Area	Gage No.	LP3 Q_{100}	Drainage Area	Gage No.	LP3 Q_{100}
0.20	9404310	237	1.23	9419590	1,080	2.72	9485550	1,920
0.24	9384200	116	1.28	9395100	345	2.79	9517200	1,240
0.30	9429510	346	1.37	9379060	301	2.85	9403800	7,350
0.32	9400200	1,520	1.38	9379100	5,880	2.94	9482480	4,460
0.35	9385800	672	1.49	9520230	2,130	3.15	9404350	18,400
0.37	9478600	417	1.61	9489080	87	3.18	9403930	708
0.44	9520110	327	1.70	9424430	2,610	3.28	9400910	182
0.45	9487140	987	1.75	9512200	3,220	3.42	9505600	573
0.46	9483040	627	1.78	9400560	770	3.53	9483045	2,260
0.51	9479200	431	1.84	9427700	1,640	3.54	9383020	913
0.64	9505900	619	1.87	9400680	413	3.57	9400530	387
0.64	9424700	993	1.98	9429150	1,270	3.63	9473200	7,490
0.65	9536350	413	1.99	9520400	3,930	3.83	9404050	449
0.66	9498600	348	1.99	9424410	1,090	4.37	9473600	1,460
0.75	9503740	220	2.04	9483200	793	4.49	9510100	2,670
0.76	9536100	589	2.06	9400660	111	4.58	9510070	5,530
0.77	9428545	296	2.08	9483250	2,870	4.72	9520130	2,380
0.79	9401245	419	2.11	9483030	7,390	4.79	9507700	2,480
0.79	9471600	385	2.15	9485950	1,090	4.93	9485900	652
0.81	9482330	560	2.18	9520160	1,620	5.22	9392800	4,030
0.83	9468300	1,690	2.30	9482950	2,390	5.25	9470900	2,140
0.85	9504100	561	2.40	9472400	6,960	5.52	9400700	326
0.90	9520300	710	2.41	9400740	293	5.57	9515800	7,450
0.95	9512420	2,910	2.43	9483025	3,360	5.57	9400580	2,220
0.95	9483010	1,210	2.43	9519600	1,670	5.88	9379560	3,530
0.98	9379980	2,850	2.44	9487400	1,300	6.01	9502700	6,250
1.07	9512700	1,730	2.55	9496800	2,850	6.31	9516600	5,330
1.15	9504400	1,430	2.56	9429400	131	6.44	9498900	4,070
1.17	9483042	842	2.60	9510170	950	6.44	9507600	8,600
1.22	9396400	1,150	2.71	9471700	2,270	6.45	9400565	2,150

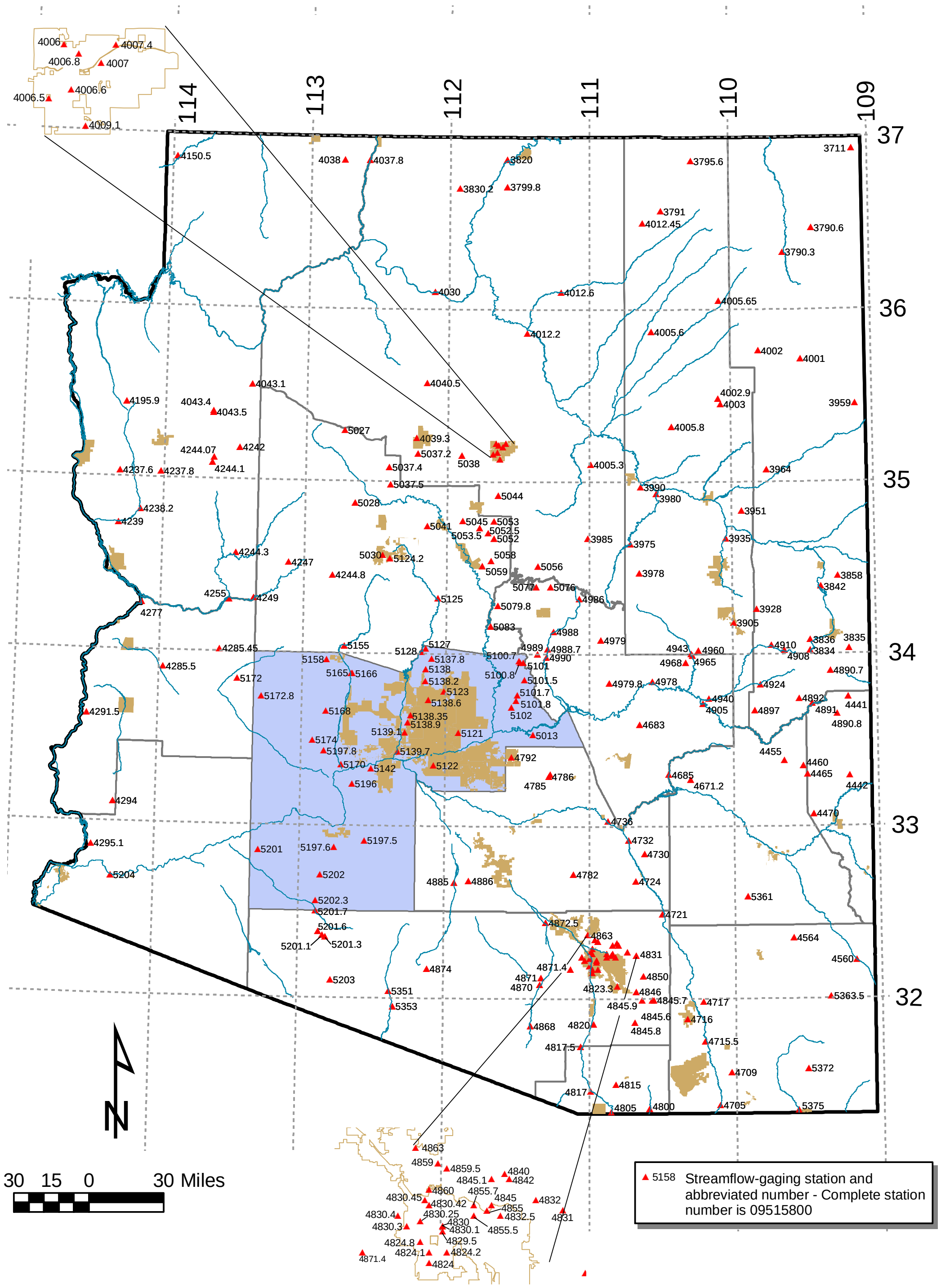
TABLE 8.1 (CONTINUED)
USGS DATA LISTING FOR WATERSHEDS WITH DRAINAGE AREAS BETWEEN .1 AND 2,000 SQUARE MILES
(ordered by increasing drainage area)

Drainage Area	Gage No.	LP3 Q_{100}	Drainage Area	Gage No.	LP3 Q_{100}	Drainage Area	Gage No.	LP3 Q_{100}
6.46	9484510	329	16.30	9484200	1,850	78.20	9491000	2,280
6.95	9424480	4,250	16.90	9383600	485	79.10	9537200	9,880
7.24	9482410	1,020	23.00	9482400	1,900	80.70	9379030	4,970
7.27	9415050	5,300	24.30	9501300	13,900	82.20	9480000	11,200
7.85	9400100	2,320	24.60	9505300	6,290	83.30	9513800	37,500
8.02	9472100	4,410	26.50	9482420	2,310	83.30	9383500	1,100
8.11	9400650	748	27.90	9397800	1,070	85.20	9517280	6,910
8.20	9483000	4,890	29.10	9383400	822	101.00	9403000	4,970
8.47	9423760	4,590	31.30	9423780	892	102.00	9445500	4,620
8.70	9520100	5,220	35.20	9467120	6,910	111.00	9505200	16,100
9.30	9400290	3,030	35.50	9484000	10,400	116.00	9519760	11,400
9.58	9485570	7,460	36.30	9503000	7,310	119.00	9489700	6,040
9.80	9510080	8,030	36.40	9508300	18,500	121.00	9512300	20,000
10.30	9481700	2,540	38.10	9489070	1,420	122.00	9498870	35,400
11.10	9513820	6,070	38.40	9484570	15,400	124.00	9503800	6,890
11.60	9444100	667	38.80	9492400	1,700	137.00	9516800	32,900
11.90	9487100	4,400	40.20	9490800	535	139.00	9512100	16,800
12.10	9520200	1,490	43.00	9483100	12,300	142.00	9505350	38,200
12.80	9488600	3,340	44.80	9485000	17,100	143.00	9424200	11,700
12.90	9519780	27,600	47.80	9517400	4,560	144.00	9478500	46,100
13.50	9424407	3,130	48.00	9505250	10,900	149.00	9446000	10,000
14.10	9484580	4,480	49.60	9400300	2,320	164.00	9510200	51,400
14.50	9503750	9,820	50.50	9484590	9,340	176.00	9481750	17,100
14.60	9428550	6,170	51.00	9400600	861	185.00	9513835	41,800
14.70	9423900	5,290	52.30	9510150	42,700	200.00	9497980	27,000
14.80	9489200	426	62.10	9497900	25,300	203.00	9496000	33,200
14.90	9503720	3,860	64.60	9513860	31,000	209.00	9481500	15,100
15.00	9456400	4,640	67.30	9513780	34,600	219.00	9484500	29,100
15.20	9510180	5,790	68.60	9390500	11,600	225.00	9494300	11,300
16.00	9371100	1,760	68.80	9519750	12,600	241.00	9505800	30,000

TABLE 8.1 (CONTINUED)
USGS DATA LISTING FOR WATERSHEDS WITH DRAINAGE AREAS BETWEEN .1 AND 2,000 SQUARE MILES
(ordered by increasing drainage area)

Drainage Area	Gage No.	LP3 Q_{100}	Drainage Area	Gage No.	LP3 Q_{100}
243.00	9520170	11,800	1439.00	9425500	69,600
250.00	9486300	23,900	1470.00	9517000	49,200
255.00	9502800	29,200	1629.00	9401260	17,300
271.00	9397500	41,000	1682.00	9482000	36,500
289.00	9484560	18,500	1730.00	9471550	28,000
295.00	9497800	21,800			
315.00	9489100	20,500			
317.00	9513890	75,100			
317.00	9398500	31,100			
323.00	9513910	47,100			
328.00	9507980	52,800			
355.00	9504500	43,700			
370.00	9404340	25,300			
377.00	9446500	24,600			
417.00	9515500	43,000			
787.00	9423820	21,200			
796.00	9516500	43,900			
814.00	9456000	8,660			
846.00	9393500	17,900			
880.00	9513970	49,000			
918.00	9486000	27,700			
1023.00	9537500	5,750			
1026.00	9468500	54,500			
1028.00	9403780	7,140			
1110.00	9512800	182,000			
1128.00	9424900	37,900			
1170.00	9487250	12,500			
1232.00	9490500	97,900			
1250.00	9535300	7,250			
1410.00	9382000	20,200			

FIGURE 8.5
LOCATIONS OF USGS GAGING STATIONS



8.4 INDIRECT METHOD NO. 3 - REGIONAL REGRESSION EQUATIONS

An analysis was performed of streamflow data for a study area comprised of Arizona, Nevada, Utah, and parts of New Mexico, Colorado, Wyoming, Texas, Idaho, Oregon, and California (USGS Open File Report 93-419, 1994). That analysis resulted in sixteen sets of regional regression equations for the study area. Two of those regions (R12 and R13) are in Maricopa County as shown in Figure 8.6. These regional regression equations can be used to estimate flood magnitude-frequencies for watersheds in Maricopa County.

Regression equations are provided for both regions to estimate flood peak discharges for frequencies of 2-, 5-, 10-, 25-, 50-, and 100-years. Use of the regression equations is recommended only if the values of the independent variables (drainage area and mean basin elevation) for the watershed of interest are within the range of the database used to derive the specific regression equation.

The regression equations for both regions (R12 and R13) are functions of drainage area. In general the equations are applicable to unregulated watersheds with drainage areas less than 200 square miles. The regression equation for Region 12 is also a function of mean basin elevation. Figure 8.7 is a scatter diagram of mean basin elevation versus drainage area for the database used to derive the regression equations as provided in USGS Open File Report 93-419.

The regression equations for Regions 12 and 13 are provided in Tables 8.2 and 8.3, respectively.

Also provided for each set of regression equations are graphs, Figures 8.8 and 8.9, of the 100-year LP3 discharge estimates versus drainage area for Flood Regions 12 and 13, respectively. A line depicting the relation between the 100-year peak discharge (computed from the regional regression equation) and drainage area is shown on each of those graphs. These graphs were recreated from the data provided in USGS Open File Report 93-419.

FIGURE 8.6
FLOOD REGIONS FOR MARICOPA COUNTY

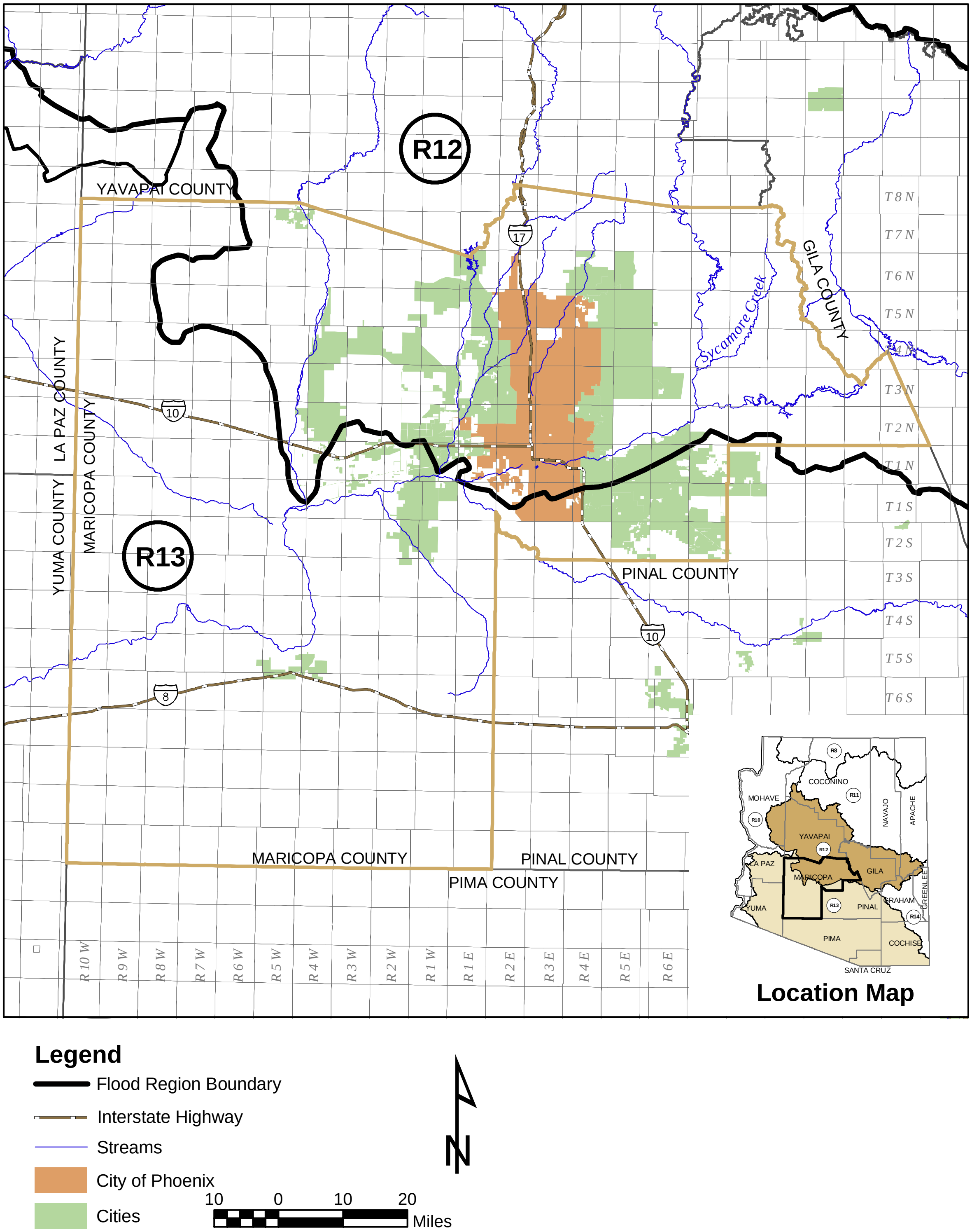


TABLE 8.2
FLOOD MAGNITUDE-FREQUENCY RELATIONS FOR THE CENTRAL ARIZONA REGION (R12)

Equation: Q, peak discharge, in cubic feet per second; $AREA$, drainage area, in square miles; and $ELEV$, mean basin elevation, in feet divided by 1,000.

Recurrence interval, in years	Equation	Average standard error of model, in percent
2	$Q = 41.1\ AREA^{0.629}$	105
5	$Q = 238\ AREA^{0.687}\ ELEV^{-0.358}$	68
10	$Q = 479\ AREA^{0.661}\ ELEV^{-0.398}$	52
25	$Q = 942\ AREA^{0.630}\ ELEV^{-0.383}$	40
50	$LOG\ Q = 7.36 - 4.17\ AREA^{-0.08} - 0.440\ LOG\ ELEV$	37
100	$LOG\ Q = 6.55 - 3.17\ AREA^{-0.11} - 0.454\ LOG\ ELEV$	39

FIGURE 8.7

SCATTER DIAGRAM OF INDEPENDENT VARIABLES FOR FLOOD REGION 12 REGRESSION EQUATION

Adapted from data contained in USGS Open File Report 93-419

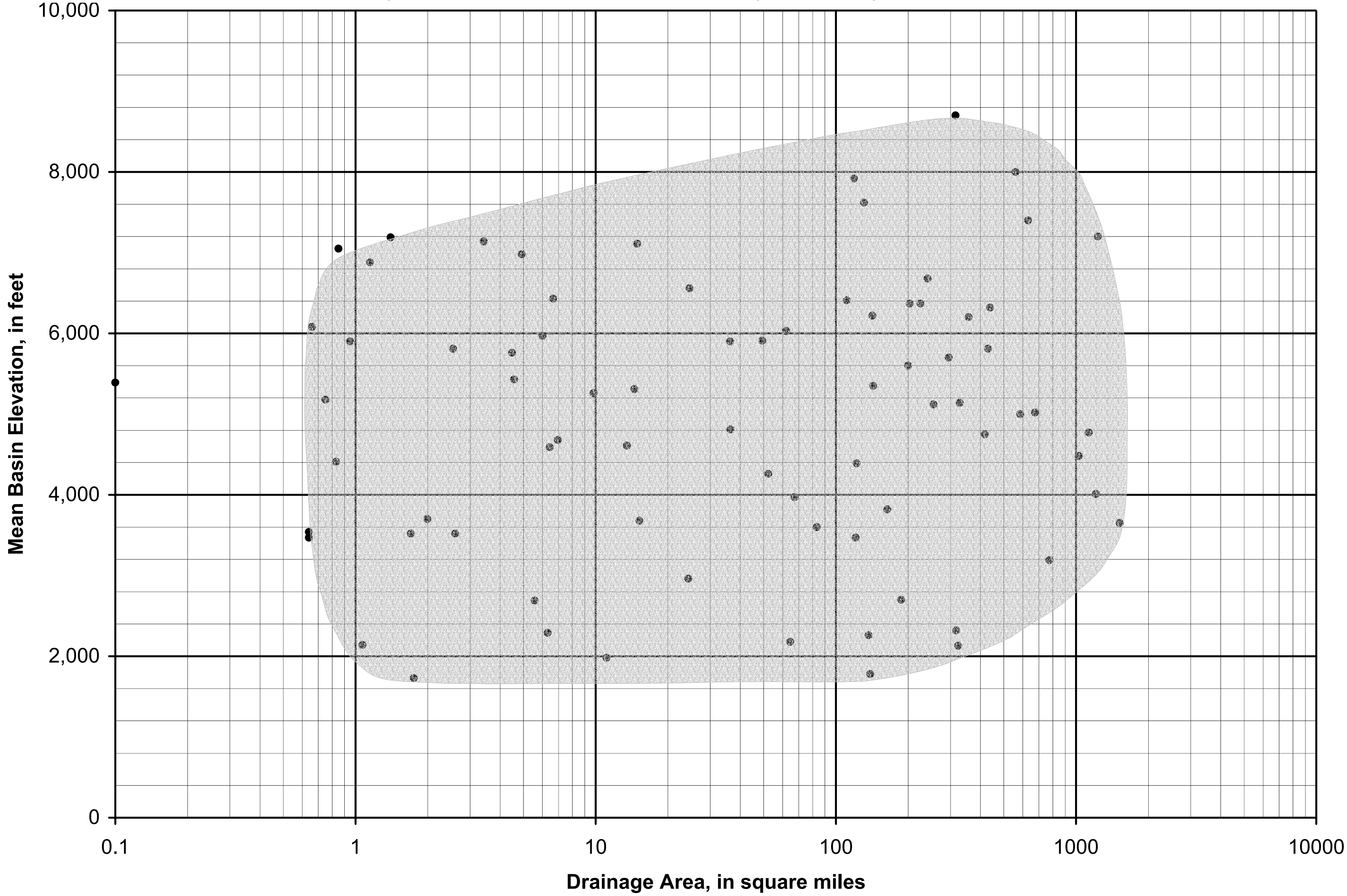


FIGURE 8.8
100-YEAR PEAK DISCHARGE RELATION FOR FLOOD REGION 12
Adapted from data contained in USGS Open File Report 93-419

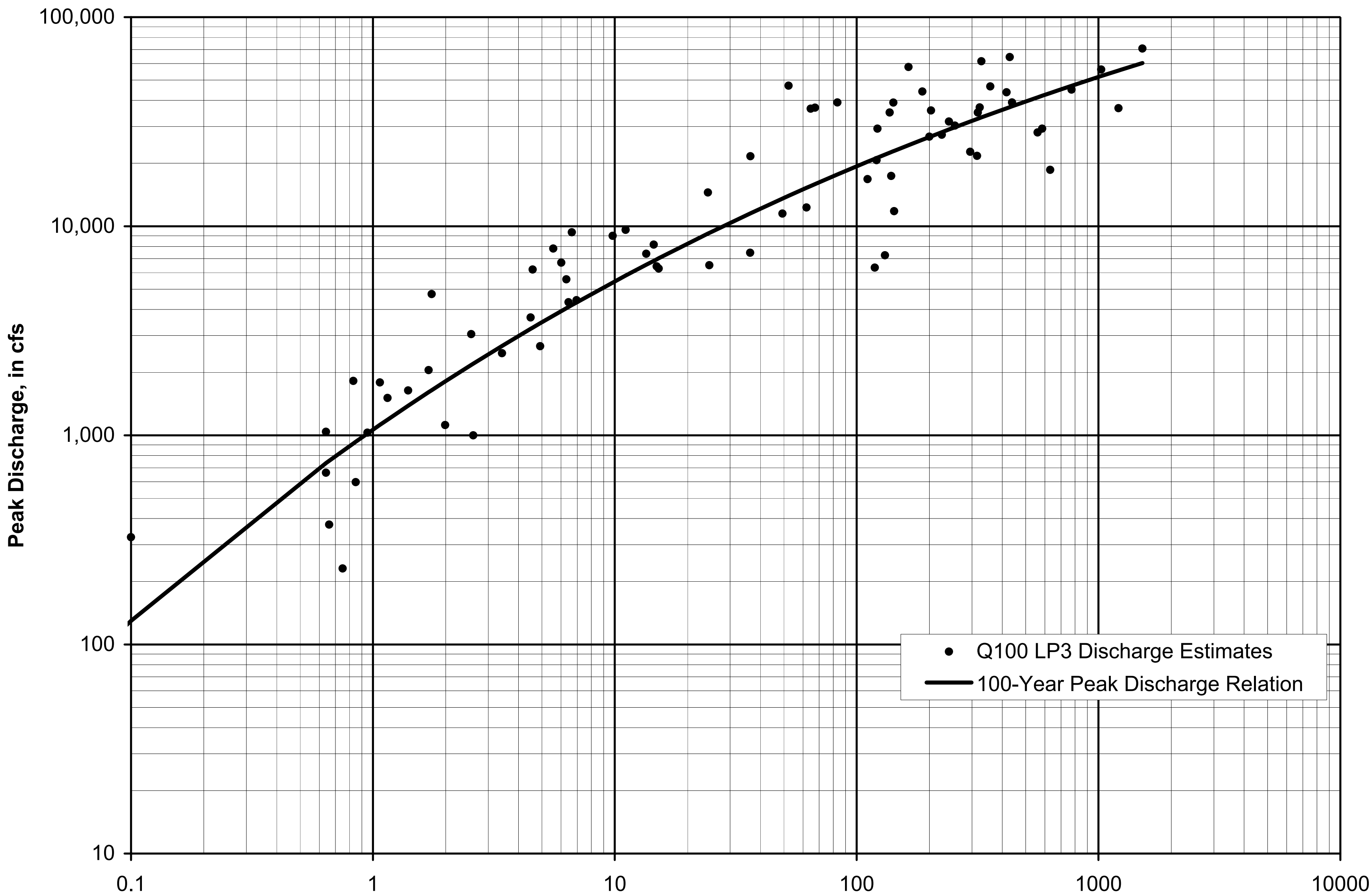


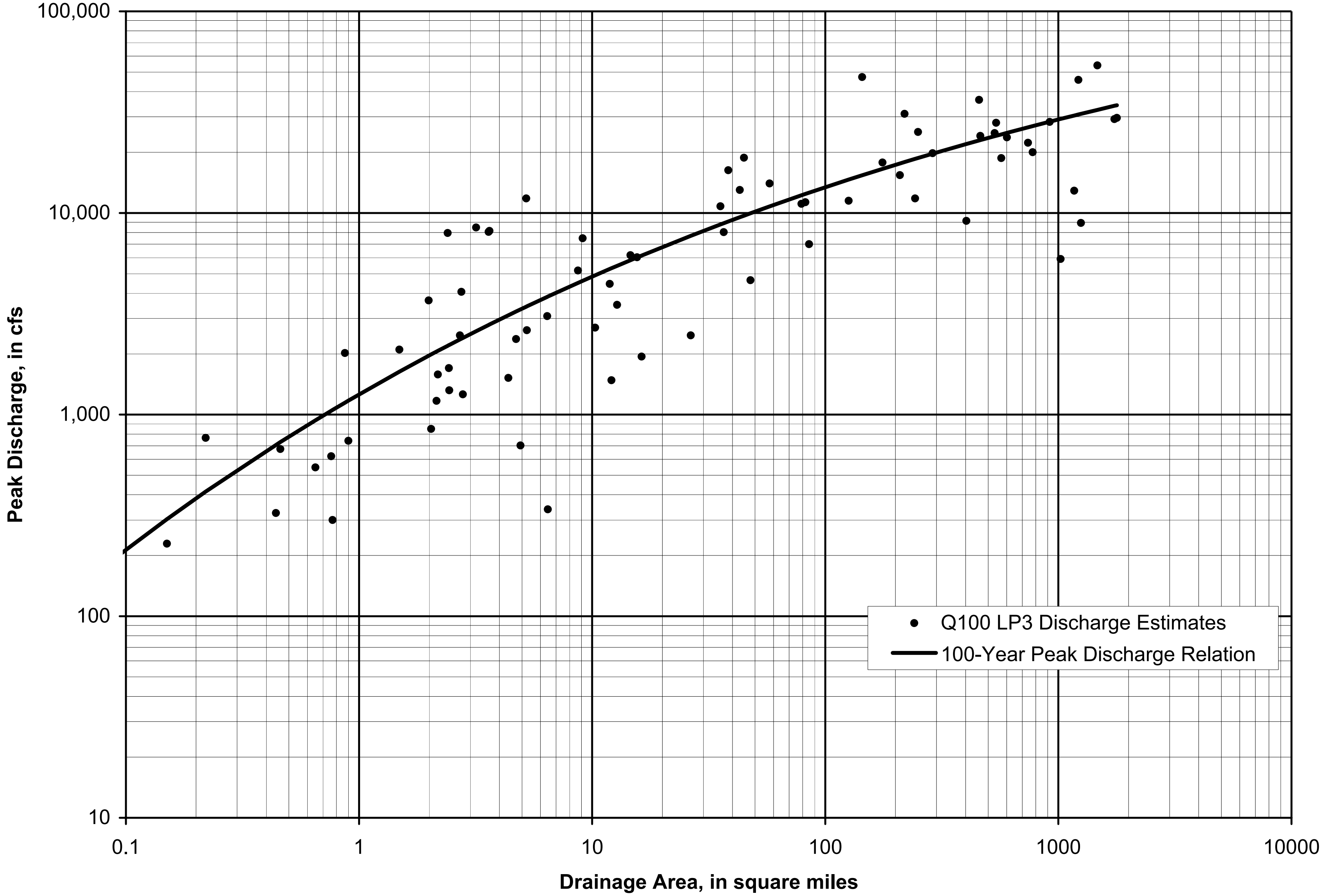
TABLE 8.3
FLOOD MAGNITUDE-FREQUENCY RELATIONS FOR THE SOUTHERN ARIZONA REGION (R13)

Equations: Q, peak discharge, in cubic feet per second; and $AREA$, drainage area, in square miles.

.

Recurrence interval, in years	Equation	Average standard error of model, in percent
2	$LOG\ Q = 6.38 – 4.29\ AREA^{-0.06}$	57
5	$LOG\ Q = 5.78 – 3.31\ AREA^{-0.08}$	40
10	$LOG\ Q = 5.68 – 3.02\ AREA^{-0.09}$	37
25	$LOG\ Q = 5.64 – 2.78\ AREA^{-0.10}$	39
50	$LOG\ Q = 5.57 – 2.59\ AREA^{-0.11}$	43
100	$LOG\ Q = 5.52 – 2.42\ AREA^{-0.12}$	48

FIGURE 8.9
100-YEAR PEAK DISCHARGE RELATION FOR FLOOD REGION 13
Adapted from data contained in USGS Open File Report 93-419



8.5 APPLICATIONS AND LIMITATIONS

The three indirect methods can be applied to any watershed in Maricopa County gaged or ungaged. Limitations exist for the use of the Regional Regression Equations based on values of the watershed characteristics as compared to the values of watershed characteristics that were used to derive these regional regression equations. The interpretation and evaluation of the results of these methods must be conducted with awareness of several factors.

1. It must be noted that these are empirical methods and the results are only applicable to watersheds that are hydrologically similar to the data base used to derive the particular method.

2. The majority of the data in all three of these methods are for undeveloped, unregulated watersheds. Urbanized watersheds can have significantly higher discharges than the results that are predicted by any of these methods.

3. These methods (other than envelope curves) produce discharge values that are statistically based averages for watersheds in the database. Conditions can exist in any watershed that would produce flood discharges, either larger than or smaller than, those indicated by these methods. Watershed characteristics that should be considered when comparing the results of indirect methods to results by analytic methods and/or flood frequency analysis are:

    a. the occurrence and extent of rock outcrop in the watershed,

    b. watershed slopes that are either exceptionally flat or steep,

    c. soil and vegetation conditions that are conducive to low rainfall losses, such as clay soils, thin soil horizons underlain by rock or clay layers, denuded watersheds (forest and range fires), and disturbed land,

    d. soil and vegetation conditions that are conducive to high rainfall losses, such as sandy soil, tilled agricultural land, and irrigated turf,

    e. land-use, especially urbanization, but also mining, large scale construction activity, and over-grazing,

    f. transmission losses that may occur in the watercourses,

    g. the existence of distributary flow areas, and

    h. upstream water regulation or diversion.

8.6 PROCEDURES

The following instructions should be followed for verifying peak discharges that are derived by analytic methods, (Rational Method or rainfall-runoff modeling). These instructions are also provided in Chapter 9, Section 9.6.

A. Verification with Unit Peak Discharge Curves:

    1. For a given watershed of drainage area (A), in square miles, divide the 100-year primary peak discharge estimate by A.

    2. Plot the unit peak discharge on a copy of Figure 8.1. Note the location of the plotted point in relation to the various curves in that figure.

B. Verification with USGS Data for Arizona:

    1. Calculate the 100-year peak discharge estimate by Equation 8.1

    2. Select Figure 8.3 or 8.4 according to watershed drainage area size, and plot the 100-year peak discharge estimate on a copy of that figure.

    3. Using watershed drainage area as a guide, identify gaged watersheds of the same approximate size from Table 8.1. Tabulate the peak discharge statistics and watershed characteristics for those gaged watersheds by using the USGS report (Garrett and Gellenbeck, 1991). Compare these to the computed peak discharge estimates and watershed characteristics for the watershed of interest.

C. Verification with Regional Regression Equations:

    1. Calculate the mean basin elevation ($ELEV$). This can be done by placing a transparent grid over the largest scale topographic map available. The grid spacing should be selected such that at least 20 elevation points are sampled. The elevation at each grid point is determined and the elevations are then averaged.

    2. Determine the flood region (Figure 8.6).

    3. Check the drainage area using the appropriate scatter diagram to determine if the values are in the “cloud of common values.” Proceed with the analysis regardless of the outcome, but clearly note if the variable values are not within the “cloud of common values.”

    4. Calculate the peak discharge estimates using the applicable regression equations for the flood region within which the project site is located.

5. Plot the 100-year peak discharge estimate on a copy of the appropriate Q_{100} data points and 100-year peak discharge relation graph (Figure 8.8 or 8.9).

D. For all three Indirect Methods:

1. Quantitatively and qualitatively analyze the results of the primary and the secondary peak discharge estimates. Address watershed characteristics that may explain differences between the primary and secondary estimates.

2. Prepare a summary of results by all methods and a qualitative evaluation of the results.

9 APPLICATION

TABLE OF CONTENTS

9.1 RAINFALL

9.1.1 Procedure for the Development of the Design Rainfall

1. Determine the size of the drainage area.

2. Determine the point rainfall depth or the areally averaged point rainfall depth, from Figures A.2 through A.13 of Appendix A, Section 1, depending on the desired storm duration and frequency.

3. For a single storm analysis, determine the depth-area reduction factor using Figure 2.1 or Table 2.1 for a 6-hour local storm and Figure 2.2 or Table 2.2 for a 24-hour general storm.

   For a multiple storm analysis, determine the drainage areas at key points of interest in the watershed. For each drainage area, determine the depth-area reduction factor using Figure 2.1 or Table 2.1 for a 6-hour local storm and Figure 2.2 or Table 2.2 for a 24-hour general storm.

4. Multiply the point rainfall depth by the appropriate depth-area reduction factor(s).

5. For a 6-hour local storm, use Figure 2.5 to select the appropriate pattern number(s) (rounded to the nearest 0.1 pattern number).

6. For a 6-hour local storm, use the dimensionless rainfall distributions of Figure 2.4 or Table 2.4 to calculate the dimensionless distribution(s) by linear interpolation between the two bounding pattern numbers.

   For a 24-hour general storm, use the dimensionless rainfall distribution of Figure 2.6 or Table 2.5.

   Note: Steps 3 through 6 are performed automatically in DDMSW.

9.1.2 User Notes

1. For a multiple storm analysis, areal reduction is accomplished in the HEC-1 program using the JD record option. The use of this record in conjunction with diversion simulations may cause an error at hydrograph combine operations downstream of the diversion. The error is that the model “looses track” of all the upstream tributary area after a diversion. Consequently the peak discharge at hydrograph combines downstream of the diversion are over estimated due to the “loss” of area. This error can be corrected by hard coding the total drainage area on the HC record of the hydrograph combine operation downstream of the diversion.

2. Use of the JD record option prohibits the use of the JR (job ratio) record option.

3. The DDMSW program automatically computes areal reduction factors and the corresponding precipitation mass curves for the 6-hour storm for a multiple storm analysis at predefined intervals. These intervals should be inspected for reasonableness in regard to the study watershed. The JD/PC records sets for storm areas greater than the next largest storm area over the total watershed area can be removed.

4. Precipitation records (PI and PC records) are coded into the HEC-1 program at the time interval specified on the IN record. The DDMSW program automatically populates these records at a time interval of 15 minutes. All other time dependent input data, such as input hydrographs (QI records) will be read into the program at the previously specified time interval unless a new time interval is specified.

9.1.3 Example

For the 22.9 sq. mile watershed, shown in Figure E.1, determine the following for a 100-year multiple storm analysis:

1. Point rainfall depth,
2. Depth-area reduction factors, and
3. Rainfall distributions.

Solution:

From Section 2.1.3, given the watershed size both the local storm (6-hour) and the general storm (24-hour) are to be considered.

1. From Figures A.7 and A.13 of Appendix A, Section 1:
   $P_6^{100} = 3.4''$
   $P_{24}^{100} = 4.4''$
2. Inspection of Figure E.1, the drainage areas of interest are:
   - Subbasin areas range from 0.44 to 5.44 sq. miles
   - Drainage areas at concentration points range from 9.83 to 22.9 sq. miles
   - Selected index areas and corresponding depth-area reduction factors from Figure 2.1 or Table 2.1 for the 6-hour storm and Figure 2.2 or Table 2.2 for the 24-hour storm are:

6-Hour Area sq. miles	6-Hour Depth-Area Reduction Factors	24-Hour Area sq. miles	24-Hour Depth-Area Reduction Factors
0.01	1.000	0.01	1.000
0.50	0.994	0.50	0.998
2.80	0.975	2.00	0.990
16.00	0.922	10.00	0.950
25.00	0.900	25.00	0.909

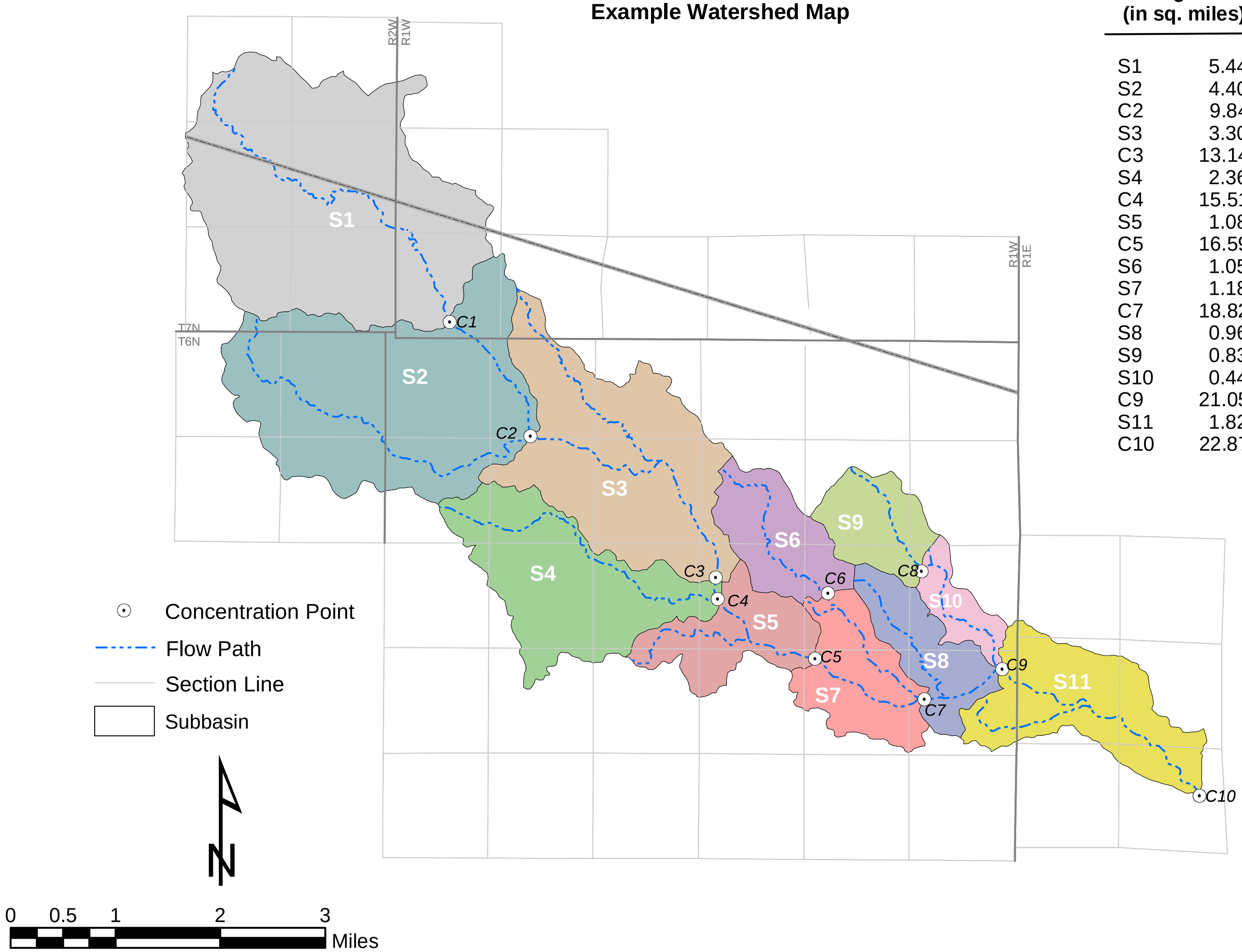
3. From Figure 2.5, the 6-hour pattern numbers corresponding to the selected index areas are 1, 2, 3 and 3.3.

Dimensionless rainfall distributions for pattern numbers 1, 2 and 3 are taken directly from Table 2.4. The distribution for pattern number 3.3 is determined by linear interpolation between pattern numbers 3 and 4 as listed in Table 2.4. The dimensionless distribution for pattern number 3.3 is:

Time hours	Pattern 3.3	Time hours	Pattern 3.3	Time hours	Pattern 3.3
0:00	0.0	2:15	13.1	4:15	78.9
0:15	1.7	2:30	14.8	4:30	86.0
0:30	2.5	2:45	16.7	4:45	90.5
0:45	3.6	3:00	19.2	5:00	94.0
1:00	5.5	3:15	24.0	5:15	95.6
1:15	7.0	3:30	32.2	5:30	97.0
1:30	8.5	3:45	48.0	5:45	98.6
1:45	10.1	4:00	66.6	6:00	100.0
2:00	11.6				

For the 24-hour storm, the SCS Type II distribution is to be used. That distribution is taken directly from Table 2.5 and is not a function of area.

Figure E.1
Example Watershed Map



Drainage Areas (in sq. miles)

S1	5.44
S2	4.40
C2	9.84
S3	3.30
C3	13.14
S4	2.36
C4	15.51
S5	1.08
C5	16.59
S6	1.05
S7	1.18
C7	18.82
S8	0.96
S9	0.83
S10	0.44
C9	21.05
S11	1.82
C10	22.87

9.2 RATIONAL METHOD

9.2.1 Procedures for the Peak Discharge Calculation

1. Determine the area within the development boundaries.

2. Select the Runoff Coefficient, C from Table 3.2. If the drainage area contains subareas of different runoff characteristics, and thus different C coefficients, arithmetically area-weight the values of C.

3. Compute the depth-duration-frequency (D-D-F) statistics for the project site using the PREFRE program (see Chapter 2, Section 2.2). Alternatively, if the project site lies within the Phoenix Metro area, then the I-D-F graph in Appendix B can be used to compute intensity.

4. Calculate the time of concentration. This is to be done as an iterative process.

   a. Determine the K_b parameter from Figure 3.1 or Table 3.1. If the drainage area contains subareas of different K_b values, arithmetically area-weight the values of K_b.

   b. Make an initial estimate of the duration and compute the intensity from the PREFRE output for the desired frequency. If the project site is within the Phoenix metro area, the I-D-F graph provided in Appendix B can be used as an alternative.

   c. Compute an estimated T_c using Equation 3.2. If the computed T_c is reasonably close to the estimated duration, then proceed to Step 5, otherwise repeat this step with a new estimate of the duration. The minimum T_c should not be less than 10-minutes.

5. Determine peak discharge Q by using the above value of i in Equation 3.1.

6. As an alternative to the above procedure, the DDMSW program may be used to calculate peak discharges.

9.2.2 Procedures for Volume Calculations

Volume calculations should be done by applying the following equation:

$$V = C\left(\frac{P}{12}\right)A \qquad (3.3)$$

where:

V = calculated volume in, acre-feet.

C = runoff coefficient from Table 3.2.

P = rainfall depth, in inches.

A = drainage area, in acres.

In the case of volume calculations for stormwater storage facility design, P equals the 100-year, 2-hour depth, in inches, as discussed in Section 2.2, and is determined from Figure A.2 of Appendix A, Section 1.

9.2.3 Procedures for the Multiple Basin Approach

The Rational Method can be used to compute peak discharges at intermediate locations within a drainage area less than 160 acres in size. A typical application of this approach is a local storm drain system where multiple subbasins are necessary to compute a peak discharge at each proposed inlet location. Consider the schematic example watershed shown in Figure E.1a. A peak discharge is needed for all three individual subareas, subareas A and B combined at Concentration Point 1 and subareas A, B and C combined at Concentration Point 2.

1. Compute the peak discharge for each individual subarea using steps 1 through 5 from Section 9.2.1.

2. Compute the arithmetically area-weighted value of C for subareas A and B.

3. Follow step 4 from Section 9.2.1 to calculate the T_c for the combined area of subareas A and B at Concentration Point 1.

4. Compare the T_c values from subareas A and B to the T_c value for the combined area at Concentration Point 1. Compute the peak discharge at Concentration Point 1 using the i for the longest T_c from step 3. If the combined peak discharge is less than the discharges for the individual subareas, use the largest discharge as the peak discharge at Concentration Point 1. The design discharge SHOULD NOT INCREASE going downstream in a conveyance system unless storage facilities are used to attenuate peak flows.

5. Compute the arithmetically area-weighted value of C for subareas A, B and C.

6. Calculate the T_c for the combined area at Concentration Point 2 using the following two methods:

Method 1 - Follow step 4 from Section 9.2.1 to calculate the T_c for the single basin composed of all three subareas.

Method 2 - Compute the travel time from Concentration Point 1 to Concentration Point 2 using the continuity equation or other appropriate technique and hydraulic parameters for the conveyance path. Add the computed travel time for the conveyance path to the T_c from Concentration Point 1.

7. Compare the T_c values from Methods 1 and 2 as well as the T_c from subarea C and calculate the peak discharge at Concentration Point 2 as follows:

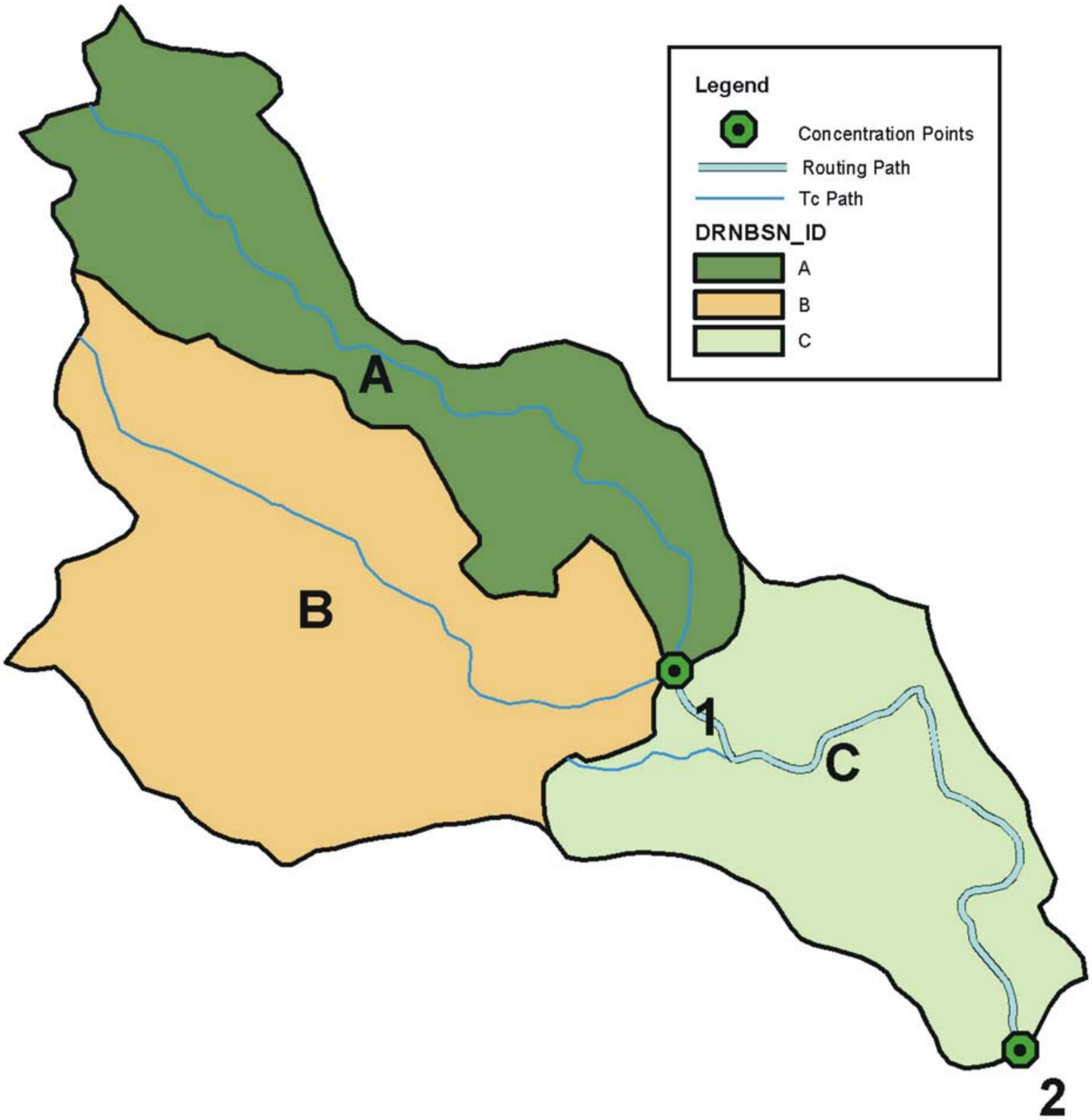
   a. If the T_c value from Method 1 is the longest, compute the total peak discharge using the Method 1 intensity, the arithmetically area-weighted value of C for all three subareas and the total contributing drainage area at Concentration Point 2.

   b. If the T_c value from Method 2 is the longest, determine i directly from the D-D-F statistics from step 3 of Section 9.2.1. Compute the total peak discharge at Concentration Point 2 using the arithmetically area-weighted value of C for all three subareas and the total contributing drainage area at Concentration Point 2.

   c. If the T_c from subarea C is the longest, compute the total peak discharge using the i for subarea C, the arithmetically area-weighted value of C for all three subareas and the total contributing drainage area at Concentration Point 2.

8. As an alternative to the above procedure, the DDMSW program may be used to calculate the peak discharge at intermediate locations.

FIGURE E.1A
SCHEMATIC EXAMPLE WATERSHED



9.2.4 User Notes

1. The Rational Method is appropriate for watersheds less than 160 acres in size.

2. For drainage areas greater than 160 acres or for situations where hydrograph routing is desired, the procedures described in Chapters 4 through 7 should be used.

3. The duration of T_c should not be longer than 2-hours and normally it will be less than 1-hour.

4, The minimum duration of T_c should not be less than 10-minutes.

5. For a multiple basin analysis, judgement must be used in the calculation of travel time, particularly in regard to velocity.

9.2.5 Example

A 37 acre mixed use residential development is planned for the tract of land as shown in Figure E.2. Off-site runoff is to be conveyed through the site in a new storm drain.

Determine the 100-year, post-development peak discharge at concentration point C1 (storm drain inlet) and C2. Also determine the total required stormwater storage volume.

- Rainfall depth-duration-frequency statistics are listed in Table E.1.
- Time of concentration physical data for each subarea are listed in Table E.2.
- Resistance coefficients for the off-site area can be characterized as moderately high for subarea S1 and moderately low for subarea S2.
- Developed areas are as follows
  - Low Density Residential = 12.0 acres
  - Medium Density Residential = 16.3 acres
  - Multiple Family Residential = 8.7 acres
- The maximum permissible velocity in the storm drain is 6 fps.
- Storm drain length = 1,700 feet.
- Assume that 10 percent of the developed areas will be needed for the local and collector roadway system.

Figure E.2
Example Watershed Map

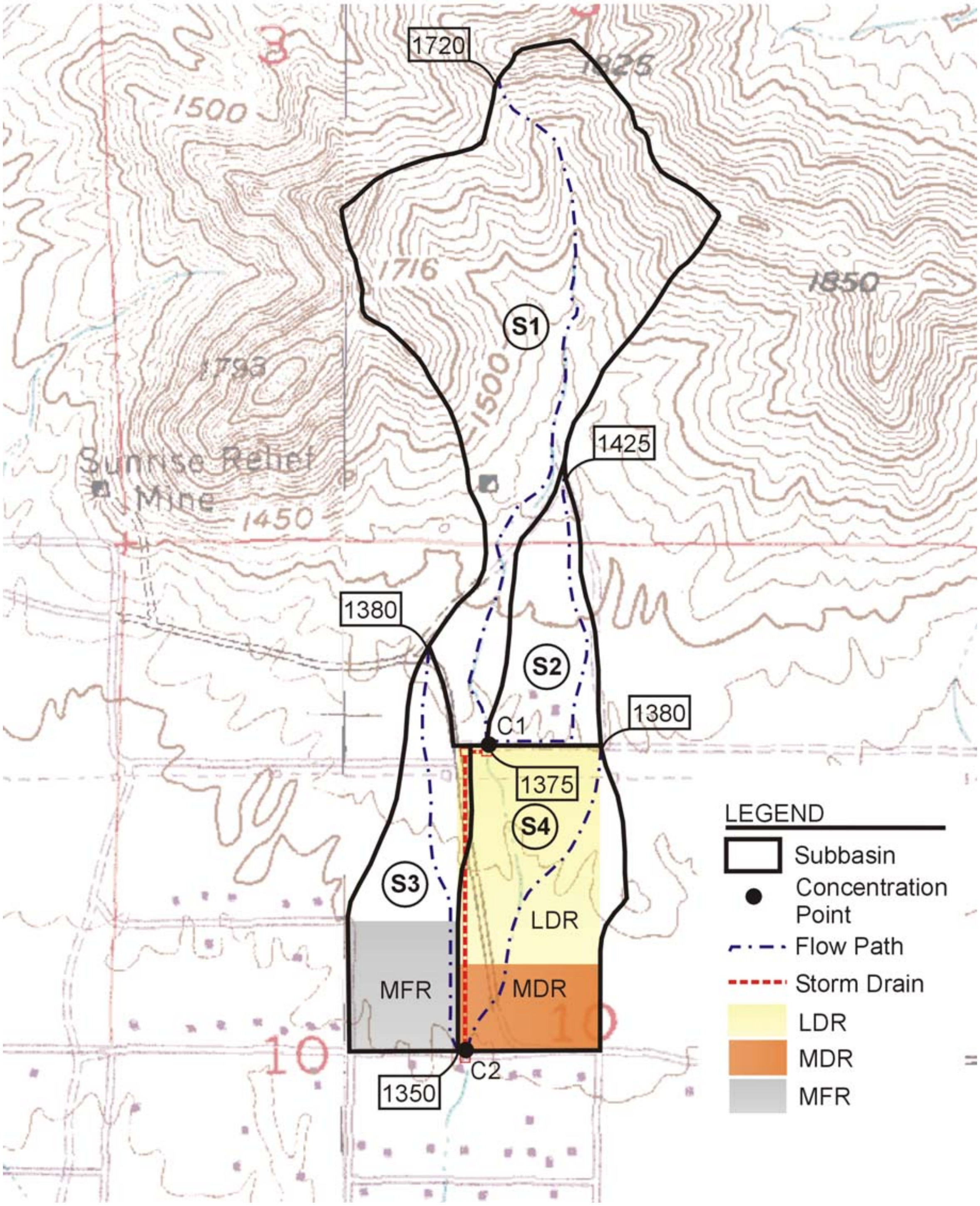


Table E.1
Depth-duration-frequency statistics

	Point Rainfall Depths in inches					
Duration	**2-Yr**	**5-Yr**	**10-Yr**	**25-Yr**	**50-Yr**	**100-Yr**
5 MIN	0.33	0.42	0.49	0.58	0.65	0.72
10 MIN	0.49	0.64	0.74	0.88	0.99	1.11
15 MIN	0.59	0.80	0.93	1.12	1.27	1.42
30 MIN	0.79	1.06	1.25	1.52	1.72	1.92
1 HOUR	0.96	1.31	1.56	1.89	2.15	2.40
2 HOUR	1.04	1.44	1.70	2.07	2.36	2.64
3 HOUR	1.10	1.52	1.80	2.19	2.50	2.80
6 HOUR	1.20	1.67	1.99	2.43	2.76	3.10
12 HOUR	1.30	1.84	2.20	2.69	3.07	3.45
24 HOUR	1.40	2.00	2.40	2.95	3.38	3.80

Note: Depth-duration-frequency statistics computed using the PREFRE program with the following input:

2-yr, 6-hr depth = 1.20
100-yr, 6-hr depth = 3.10
2-yr, 24-hr depth = 1.40
100-yr, 24-hr depth = 3.80

Table E.2
Time of concentration physical data

Subbasin ID (1)	**Drainage Area acres (2)**	**Flow Path Length miles (3)**	**Flow Path Slope ft/mi (4)**
S1	66.0	0.725	475.6
S2	12.4	0.367	136.1
S3	22.6	0.407	73.7
S4	28.3	0.337	89.0
	129.3		

Solution:

1. Select the Runoff Coefficients (C) for each land use from Table 3.2
   - Mountain Terrain (NMT); $C = 0.75$
   - Undeveloped Desert Rangeland (NDR); $C = 0.50$
   - Low Density Residential (LDR); $C = 0.55$
   - Medium Density Residential (MDR); $C = 0.75$
   - Multiple Family Residential (MFR); $C = 0.85$
   - Pavement (P); $C = 0.95$

   Compute the arithmetically area-weighted C value for subareas S3 and S4

   - Subarea S3:

$$C_w = \frac{(0.5)13.9 + (0.85)7.8 + (0.95)0.9}{22.6} = 0.64$$

   - Subarea S4:

$$C_w = \frac{(0.55)14.7 + (0.75)10.8 + (0.95)2.8}{28.3} = 0.67$$

   The Runoff Coefficients for each subarea are

   - S1; $C = 0.75$
   - S2; $C = 0.50$
   - S3; $C = 0.64$
   - S4; $C = 0.67$

2. Compute the Resistance Coefficient (K_b) for each subarea using Table 3.1.

- Subarea S1: using K_b type C

$$K_b = -0.025 \log(66.0) + 0.15 = 0.105$$

- Subarea S2: using K_b type B

$$K_b = -0.01375 \log(12.4) + 0.08 = 0.065$$

- Subarea S3: using K_b type B for the off-site area and K_b type A for the on-site area

$$K_{b(off)} = -0.01375 \log(13.9) + 0.08 = 0.064$$

$$K_{b(on)} = -0.00625 \log(8.7) + 0.04 = 0.034$$

$$K_{b(weighted)} = \frac{(0.064)13.9 + (0.034)8.7}{22.6} = 0.053$$

- Subarea S4: using K_b type A

$$K_b = -0.00625 \log(28.3) + 0.04 = 0.031$$

The K_b values for each subarea are

- S1; $K_b = 0.105$
- S2; $K_b = 0.065$
- S3; $K_b = 0.053$
- S4; $K_b = 0.031$

3. Compute the Time of Concentration (T_c) and Intensity (i) for each subarea using Equation 3.2.

- Subarea S1

$$T_c = 11.4\,L^{0.5}\,K_b^{0.52}\,S^{-0.31}\,i^{-0.38}$$
$$= 11.4\,(0.725)^{0.5}(0.105)^{0.52}(475.6)^{-0.31}\,i^{-0.38}$$
$$= 0.445\,i^{-0.38}$$

Start with an initial estimate for T_c of 15 min.
- From Table E.1, $i = 1.42/(15/60) = 5.68$ in/hr
- $T_c = 0.445\,(5.68)^{-0.38} = 0.230$ hrs (13.8 min.)

Recompute i for $T_c = 13.8$ min.
- From Table E.1, $i = 1.346/(13.8/60) = 5.85$ in/hr
- $T_c = 0.445\,(5.85)^{-0.38} = 0.227$ hrs (13.6 min.)

Recompute i for $T_c = 13.6$ min
- From Table E.1, $i = 1.333/(13.6/60) = 5.88$ in/hr
- $T_c = 0.445\,(5.88)^{-0.38} = 0.227$ hrs

Use $T_c = 13.6$ min; $i = 5.88$ in/hr

Using the above procedure, the T_c and i for each subarea are

Subarea	T_c min	i in/hr
S1	13.6	5.88
S2	10.7	6.47
S3	12.6	6.05
S4	10.0	6.66

4. Compute the peak discharge for each subarea using Equation 3.1: $Q = CiA$

   - Subarea S1 : $Q = (0.75)(5.88)(66.0) = 291$ cfs
   - Subarea S2 : $Q = (0.5)(6.47)(12.4) = 40$ cfs
   - Subarea S3 : $Q = (0.64)(6.05)(22.6) = 88$ cfs
   - Subarea S4 : $Q = (0.67)(6.66)(28.3) = 126$ cfs

5. Compute the peak discharge at concentration point C1, the storm drain inlet.

   a. Compute the arithmetically area-weighted C value of subareas S1 and S2.

   $$C_w = \frac{(0.75)66.0 + (0.5)12.4}{78.4} = 0.71$$

   b. Compute the arithmetically area-weighted K_b value of subareas S1 and S2.

   $$K_b = \frac{(0.105)66.0 + (0.065)12.4}{78.4} = 0.099$$

   c. Compute the T_c and i using Equation 3.2

   $$\begin{aligned} T_c &= 11.4 L^{0.5} K_b^{0.52} S^{-0.31} i^{-0.38} \\ &= (11.4)(0.725)^{0.5}(0.099)^{0.52}(475.6)^{-0.31} i^{-0.38} \\ &= 0.431 i^{-0.38} \end{aligned}$$

   Start with an initial estimate for T_c of 13 min
   - From Table E.1, $i = 1.296/(13/60) = 5.98$ in/hr
   - $T_c = 0.431(5.98)^{-0.38} = 0.218$ hrs (13.1 min)

Recompute i for $T_c = 13.1$ min.
- From Table E.1, $i = 1.302/(13.1/60) = 5.96$ in/hr
- $T_c = 0.431\,(5.96)^{-0.38} = 0.219$ hrs (13.1 min)

Use $T_c = 13.1$ min, $i = 5.96$ in/hr

d. Compute the peak discharge at concentration point C1 using Equation 3.1 and the longest T_c.

The longest T_c is for subarea S1, 13.6 min.

$Q = CiA = (0.71)(5.88)(78.4) = 327$ cfs

6. Compute the peak discharge at concentration point C2

a. Compute the arithmetically area-weighted C value for subareas S1, S2, S3 and S4.

$$C_w = \frac{(0.75)66.0 + (0.5)(12.4) + (0.64)22.6 + (0.67)28.3}{129.3} = 0.69$$

b. Compute the arithmetically area-weighted K_b value for subareas S1, S2, S3 and S4.

$$K_b = \frac{(0.105)66.0 + (0.065)12.4 + (0.053)22.6 + (0.031)28.3}{129.3}$$

$K_b = 0.076$

c. Compute the T_c and i using Equation 3.2 and the Method 1 approach

$L = 0.725 + (1700/5280) = 1.047$ mi
$S = 350.9$ ft/mi

$$T_c = 11.4 L^{0.5} K_b^{0.52} S^{-0.31} i^{-0.38}$$
$$= (11.4)(1.047)^{0.5}(0.076)^{0.52}(350.9)^{-0.31} i^{-0.38}$$
$$= 0.496\, i^{-0.38}$$

Start with an initial estimate for T_c of 15 min.
- From Table E.1, $i = 1.42/(15/60) = 5.68$ in/hr
- $T_c = 0.496(5.68)^{-0.38} = 0.256$ hrs (15.4 min)

Recompute i for $T_c = 15.4$ min.
- From Table E.1, $i = 1.433/(15.4/60) = 5.58$ in/hr
- $T_c = 0.496(5.58)^{-0.38} = 0.258$ hrs (15.5 min)

Recompute i for $T_c = 15.5$ min.
- From Table E.1, $i = 1.437/(15.5/60) = 5.56$ in/hr
- $T_c = 0.496(5.56)^{-0.38} = 0.258$ hrs (15.5 min)

Use $T_c = 15.5$ min, $i = 5.56$ in/hr

d. Compute the T_c and i using Equation 3.2 and the Method 2 approach

Travel time in the storm drain from C1 to C2 assuming a maximum velocity of 6 fps

$$T_t = 1700/(6*60) = 4.7 \text{ min}$$

Total time at concentration point C2

$$T_c = T_c^{c1} + T_t = 13.6 + 4.7 = 18.3 \text{ min}$$

From Table E.1 with a duration of 18.3 min

$$i = 1.530/(18.3/60) = 5.02 \text{ in/hr}$$

e. Compute the peak discharge at concentration point C2 using Equation 3.1 and the longest T_c

The longest T_c is for the Method 2 approach

$$Q = CiA = (0.69)(5.02)(129.3) = 448 \text{ cfs}$$

7. Compute the total required stormwater storage volume using Equation 3.3

- From Table E.1, $P = 2.64$ inches
- Compute the arithmetically area-weighted C value

$$C_w = \frac{(0.85)7.8 + (0.75)14.7 + (0.55)10.8 + (0.95)3.7}{37} = 0.73$$

$$V = CPA = (0.73)\left(\frac{2.64}{12}\right)(37) = 5.9 \text{ AF}$$

9.3 RAINFALL LOSSES

9.3.1 Procedures for the Green and Ampt Method

A. When soils data are available:

1. Prepare a base map of the drainage area delineating subbasins, if used.

2. Determine the location of the study area in regard to the limits of the soil surveys provided in Appendix C.

    a. If the study area is completely contained within these limits:

        i. Overlay the watershed limits on the soil survey maps from the appropriate soil survey report(s) and tabulate the map units present within the watershed.

        ii. Cross reference the map units with those listed in Appendix C and tabulate the weighted value of XKSAT for each map unit and the corresponding percent imperviousness.

        iii. Proceed to item (3) or (4).

    b. If the study area is partly or entirely outside the limits of the soils surveys provided in Appendix C:

        i. Refer to the figure showing the status of soil surveys in Arizona (at the front of Appendix C) for other sources of soils data. Other sources of soils data are:

            - General soils surveys by county prepared by the NRCS.
            - Other detailed soil surveys.
            - Terrestrial Ecosystem Survey of Tonto National Forest.

        ii. Using the data contained in the alternative source, follow the example procedure for determination of the weighted XKSAT value for each unique map unit that is included at the front of Appendix C.

        iii. Proceed to item (3) or (4).

3. If the watershed or subbasin contains only one soil texture, then determine XKSAT, PSIF and DTHETA from Table 4.1.

4. If the watershed or subbasin is composed of soils of different textures, then area-weighted parameter values will be calculated:

    a. Calculate the area-weighted value of XKSAT by using Equation 4.4.

    b. Select the corresponding values of PSIF and DTHETA from Figure 4.3.

    c. Calculate the arithmetically area-weighted value of naturally occurring RTIMP.

5. Select values of IA for each land use and/or soil cover using Table 4.2. Arithmetically area-weight the values of IA if the drainage area or subbasin is composed of subareas of different IA.

6. Select values of RTIMP for each land use using Table 4.2. Arithmetically area-weight the values of RTIMP if the drainage area or subbasin is composed of land use subareas of different RTIMP. Compute the total weighted value of RTIMP based on the area-weighted land use and naturally occurring RTIMP.

7. Estimate the vegetative cover (VC) for the natural portions of the drainage area or subbasin. Select values of VC for each land use using Table 4.2. Arithmetically area-weight the values of VC if the drainage area or subbasin is composed of land use subareas of different VC. Arithmetically average the natural VC and the area-weighted land use VC.

8. Adjust the XKSAT value for VC using Figure 4.4, if appropriate.

9. Arithmetically average $DTHETA_{dry}$ (natural portions of the drainage area or subbasin) and $DTHETA_{normal}$ (Developed portions of the drainage area or subbasin), if appropriate.

B. Alternative Methods:

As an alternative to the above procedures, Green and Ampt loss rate parameters can be estimated by reconstitution of recorded rainfall-runoff events on the drainage area or hydrologically similar watersheds, or parameters can be estimated by use of rainfall simulators in field experiments. Plans and procedures for estimating Green and Ampt loss rate parameters by either of these procedures should be approved by the Flood Control District and/or the local agency before initiating the procedures.

9.3.2 Procedures for the Initial Loss Plus Uniform Loss Rate Method

A. When soils data are available:

1. Prepare a base map of the drainage area delineating modeling subbasins, if used.
2. Delineate subareas of different infiltration rates (uniform loss rates) on the base map. Assign a land-use or surface cover to each subarea.
3. Determine the size of each subbasin and size of each subarea within each subbasin.
4. Estimate the impervious area (RTIMP) for the drainage area or each subarea.
5. Estimate the initial loss (STRTL) for the drainage area or each subarea by regional studies or calibration. Alternatively, Equation 4.5 or Tables 4.2 and 4.4 can be used to estimate or to check the value of STRTL.
6. Estimate the uniform loss rate (CNSTL) for the drainage area or each subarea by regional studies or calibration. Table 4.3 can be used, in certain situations, to estimate or to check the values of CNSTL.
7. Calculate the area-weighted values of RTIMP, STRTL, and CNSTL for the drainage area or each subbasin.
8. Enter the area-weighted values of RTIMP, STRTL, and CNSTL for the drainage area or each subbasin on the LU record of the HEC-1 input file.

9.3.3 User Notes

1. There are currently six soil survey volumes available for Maricopa County and adjoining areas. Five of these are published by the National Resource Conservation Service (NRCS). A figure showing the status and extent of each NRCS survey is provided at the front of Appendix C. Copies of these survey reports can be obtained from the NRCS field offices. Data from three of these surveys have been summarized and are included in Appendix C, Sections 2, 3 and 4 along with map unit values of XKSAT and rock outcrop percentages. The sixth soil survey is published by the Forest Service and is entitled Tonto National Forest Terrestrial Ecosystem Survey. A copy of this survey can also be obtained from the Forest Service field office.
2. Map unit values of XKSAT (bare ground) are calculated based on individual soil textures in a map unit, percentages of soil textures in a map unit, XKSAT values from Table 4.1, and a logarithmic area-weighting procedure. Since many of the soil groups contain horizons of different textures, the top texture may or may not control the total

volume and rate of infiltration. The decision of which soil layer controls the infiltration rate is based on soil texture, horizon thickness, and the accumulated depth of water during the initial low intensity period of a design storm.

3. Impervious cover percentages, applied in an HEC-1 model using the RTIMP variable, directly converts the assigned percentage of areal rainfall to runoff. This assumes that the impervious area is hydraulically connected to the outlet. Impervious cover percentages (i.e. rock outcrop) listed in the soil surveys may or may not be hydraulically connected to the outlet. Judgement should be exercised in the assignment of the effectiveness of impervious cover percentages based on the soil surveys.

4. The PSIF and DTHETA values are taken from Figure 4.3 as a function of the basin or subbasin average value of XKSAT (bareground) not for each map unit value of *XKSAT.*

5. XKSAT (bareground) is adjusted for the effects of vegetation cover by use of Figure 4.4. The PSIF and DTHETA values are not a function of the adjusted XKSAT value and are not adjusted for vegetation cover.

6. For a partially developed basin or subbasin, DTHETA dry and DTHETA normal can be readily averaged based on the percentage of the natural and developed area.

7. The DTHETA “Saturated” condition should be used only if the entire area is under irrigation simultaneously.

9.3.4 Example

Compute the area-weighted Green and Ampt rainfall loss parameters for each subbasin shown in Figure E.1 (see Chapter 2, Section 2.7). Soil map units as they occur within the watershed are shown in Figure E.3. The majority of the watershed lies within the limits of the Soil Survey of Aguila - Carefree and Parts of Maricopa and Pinal Counties, Arizona. The remaining portion of the watershed lies within the limits of the Soil Survey of Yavapai County, Arizona, Western Part. Soil characteristics for each map unit are provided in Table E.3. The area of each map unit present within each subbasin is provided in Table E.4 along with the corresponding soil characteristics. Vegetation cover for all natural portions of the watershed is estimated to be 26 percent. Developed areas within the watershed are shown in Figure E.4. Land use characteristics are provided in Table E.5. The area of each land use type present within each subbasin is provided in Table E.6 along with the corresponding land use characteristics.

Solution:

1. Compute the log-averaged bare ground XKSAT for each subbasing using Equation 4.4.

   - Subbasin S1 : $A_T = 3{,}481.6$ acres

   Log-averaged XKSAT $= 10^{a}$; where :

$$a = \left[\frac{(5.6)\log 0.96 + (176.8+856.1)\log 0.44 + (1723.5)\log(0.33) + (143.9+53.0)\log 0.09 + (74.5+410.6)\log 0.14 + (37.6)\log 0.01}{3{,}481.6}\right]$$

$$a = \frac{-1893.5}{3481.6} = -0.54$$

Log-averaged XKSAT $= 10^{-0.54} = 0.29$ in/hr

- Using the above procedure, the log-averaged XKSAT for each subbasin is as follows:

Subbasin ID	Log-Averaged XKSAT in/hr
S1	0.29
S2	0.33
S3	0.20
S4	0.30
S5	0.32
S6	0.14
S7	0.23
S8	0.21
S9	0.11
S10	0.13
S11	0.24

2. From Figure 4.3, select the values for DTHETA (both dry and normal) and PSIF for each subbasin corresponding to the computed XKSAT from (1).

Subbasin ID	Log-Average XKSAT in/hr	DTHETA Dry	DTHETA Normal	PSIF inches
S1	0.29	0.35	0.25	4.66
S2	0.33	0.35	0.25	4.34
S3	0.20	0.37	0.25	5.51
S4	0.30	0.35	0.25	4.58
S5	0.32	0.35	0.25	4.41
S6	0.14	0.39	0.23	6.43
S7	0.23	0.36	0.25	5.15
S8	0.21	0.37	0.25	5.20
S9	0.11	0.36	0.17	6.92
S10	0.13	0.38	0.21	6.62
S11	0.24	0.35	0.25	5.05

3. Compute the arithmetically area-weighted surface retention loss (IA) for each subbasin for natural conditions.

- Subbasin S1 : $A_T = 3{,}481.6$ acres

$$IA_w = \left[\frac{(5.6)(0.35) + (176.8 + 1723.5 + 143.9 + 74.5 + 856.1 + 53.0 + 410.6 + 37.6)(0.25)}{3481.6}\right]$$

$$IA_w = \frac{871.0}{3481.6} = 0.25 \text{ inches}$$

- Using the above procedure, the area-weighted IA for each subbasin is as follows:

Subbasin ID	IA_N inches
S1	0.25
S2	0.25
S3	0.26
S4	0.25
S5	0.26
S6	0.25
S7	0.24
S8	0.20
S9	0.27
S10	0.24
S11	0.18

4. Compute the arithmetically area-weighted percent impervious (RTIMP) for each subbasin for natural conditions.

- Subbasin S1 : A_T = 3,481.6 acres

$$RTIMP_W = \left[\frac{(5.6)(0) + (176.8 + 856.1)(15) + (1723.5)(35) + (143.9 + 53.0)(30) + (74.5 + 410.6)(60) + (37.6)(50)}{3481.6}\right]$$

$$RTIMP_W = \frac{112,709}{3481.6} = 32\,\%$$

- Using the above procedure, the area-weighted RTIMP for each subbasin is as follows:

Subbasin ID	$RTIMP_N$ %
S1	32
S2	35
S3	33
S4	34
S5	34
S6	55
S7	21
S8	22
S9	39
S10	37
S11	3

5. Compute the arithmetically area-weighted RTIMP for each subbasin for developed conditions (the developed portion of each subbasin only).

- Subbasin S3 : A_D = 1,993.1 acres

$$RTIMP_w = \left[\frac{(117.0)(80)+(699.5)(15)+(129.3)(30)+(1037.3)(5)}{1993.1}\right]$$

$$RTIMP_w = \frac{28918}{1993.1} = 15\ \%$$

- Using the above procedure, the area-weighted RTIMP for the developed portion of each subbasin is as follows:

Subbasin ID	$RTIMP_D$ %
S1	0
S2	15
S3	15
S4	20
S5	18
S6	8
S7	21
S8	36
S9	5
S10	15
S11	29

6. Compute the arithmetically area-weighted vegetation cover (VC) for each subbasin for developed conditions.

- S3 : A_D = 1,993.1 acres

$$VC_w = \left[\frac{(117.0)(15) + (699.5)(43) + (10)(80) + (129.3)(35) + (1037.3)(29)}{\cancel{1991.3}\ 1993.1}\right]$$

$$VC_w = \frac{67241}{1993.1} = 34\ \%$$

- Using the above procedure, the area-weighted VC for the developed portion of each subbasin is as follows:

Subbasin ID	VC_D %
S1	0
S2	43
S3	34
S4	40
S5	40
S6	33
S7	41
S8	33
S9	29
S10	43
S11	36

7. Compute the arithmetically area-weighted surface retention loss (IA) for each subbasin for the developed portion.

- Subbasin S3 : $A_D = 1{,}993.1$

$$IA_w = \left[\frac{(117.0)(0.10) + (699.5)(0.30) + (10)(0.20) + (129.3)(\cancel{25}\,0.25) + (1037.3)(0.30)}{1993.1}\right]$$

$$IA_w = \frac{566.1}{1993.1} = 0.28 \text{ inches}$$

- Using the above procedure, the area-weighted IA for the developed portion of each subbasin is as follows:

Subbasin ID	IA inches
S1	0
S2	0.30
S3	0.28
S4	0.28
S5	0.29
S6	0.30
S7	0.28
S8	0.23
S9	0.30
S10	0.30
S11	0.25

8. Compute the average values of DTHETA, IA, RTIMP and VC for each subbasin based on the percent developed and percent natural areas.

- Subbasin S2 : A_T = 2,816.9 acres

$$\text{Natural Area} = \frac{1753.9}{2816.9} = 62\,\%$$

$$\text{Developed Area} = 38\,\%$$

$$\begin{aligned} DTHETA_{AVG} &= \left((62)\,DTHETA_{Dry} + (38)\,DTHETA_{Normal}\right)/100 \\ &= \left((62)(0.35) + (38)(0.25)\right)/100 \\ &= 0.31 \end{aligned}$$

$$\begin{aligned} IA_{AVG} &= (62)IA_N + (38)\,IA_D \\ &= \left((62)(0.25) + (38)(0.30)\right)/100 \\ &= 0.27 \text{ inches} \end{aligned}$$

$$RTIMP_{AVG} = RTIMP_N + \left(1 - \frac{RTIMP_N}{100}\right)(38)\frac{RTIMP_D}{100}$$
$$= 35 + \left(1 - \frac{35}{100}\right)(38)\left(\frac{15}{100}\right)$$
$$= 39\%$$

$$VC_{AVG} = \left((62)VC_N + (38)VC_D\right)/100$$
$$= \left((62)(26) + (38)(43)\right)/100$$
$$= 33\%$$

- Using the above procedure, the average values of DTHETA, IA, RTIMP and VC for each subbasin are as follows:

Subbasin ID	DTHETA	IA inches	RTIMP %	VC %
S1	0.35	0.25	32	26
S2	0.31	0.27	39	33
S3	0.26	0.28	43	33
S4	0.25	0.28	48	40
S5	0.26	0.28	45	38
S6	0.25	0.29	58	32
S7	0.29	0.27	31	35
S8	0.29	0.22	40	31
S9	0.19	0.30	41	29
S10	0.21	0.30	46	43
S11	0.33	0.19	9	28

9. Compute the vegetion cover correction factor using Figure 4.4 and the adjusted XKSAT for each subbasin

Subbasin ID	Correction Factor	Adjusted XKSAT in/hr
S1	1.18	0.34
S2	1.26	0.41
S3	1.26	0.25
S4	1.33	0.39
S5	1.31	0.42
S6	1.24	0.17
S7	1.28	0.30
S8	1.23	0.26
S9	1.21	0.13
S10	1.37	0.17
S11	1.20	0.29

10. The area-weighted Green and Ampt rainfall loss parameters for each subbasin are as follows:

Subbasin ID	IA inches	DTHETA	PSIF inches	XKSAT in/hr	RTIMP %
S1	0.25	0.35	4.66	0.34	32
S2	0.27	0.31	4.34	0.41	39
S3	0.28	0.26	5.51	0.25	43
S4	0.28	0.25	4.58	0.39	48
S5	0.28	0.26	4.41	0.42	45
S6	0.29	0.25	6.43	0.17	58
S7	0.27	0.29	5.15	0.30	31
S8	0.22	0.29	5.20	0.26	40
S9	0.30	0.19	6.92	0.13	41
S10	0.30	0.21	6.62	0.17	46
S11	0.19	0.33	5.05	0.29	9

Table E.3
Rainfall loss characteristics for each soil map unit

Map Unit ID (1)	Description (2)	XKSAT[1] in/hr (3)	RTIMP[1] % (4)	IA[2] inches (5)
8	Very cobbly sandy loam	0.96	0	0.35
10	Loamy sand	0.94	0	0.35
16	Very gravelly fine sandy loam	0.44	15	0.25
21	Very gravelly loam	0.38	0	0.35
31	Extremely cobbly sandy loam	0.33	35	0.25
33	Very gravelly loam	0.23	0	0.35
41	Very gravelly loam	0.17	0	0.25
45	Very gravelly clay	0.03	0	0.25
48	Very gravelly clay	0.06	0	0.15
51	Very gravelly sandy clay loam	0.24	0	0.15
52	Very gravely clay loam	0.16	20	0.25
66	Very gravely loam	0.23	0	0.35
68	Very gravely sandy loam	0.63	0	0.35
70	Very gravely loam	0.36	0	0.25
72	Clay loam	0.09	30	0.25
93	Gravelly loam	0.33	0	0.25
95	Clay loam	0.04	0	0.35
103	Very gravelly clay loam	0.10	65	0.25
104	Gravelly clay loam	0.14	60	0.25
108	Very cobbly loam	0.31	30	0.25
109	Very cobbly loam	0.35	35	0.25
CmD	Very gravelly sandy loam	0.44	15	0.25
Le	Gravelly clay loam	0.09	30	0.25
Lh	Extremely rocky clay loam	0.14	60	0.25
Rr	Rock outcrop	0.01	50	0.25

Notes:

1. Values for the soil map units within the limits of the Soil Survey of Aguila-Carefree and Parts of Maricopa and Pinal Counties, Arizona are taken from Appendix C, Seciton 1. Values for the soil map units within the limits of the Soil Survey of Yavapai County, Arizona, Western Part are based on a comparison of the soil texture descriptions to those of the adjacent soil map units in the Aguila-Carefree Soil Survey.
2. Values are based on the descriptions in the soil surveys and the use of Table 4.2.

Table E.4
Summary of soils characteristics for each subbasin

Subbasin ID	Map Unit	Map Unit Area sq. feet	Map Unit Area acres	Map Unit Area sq. miles	XKSAT in/hr	RTIMP %	IA inches
S1	8	244991	5.6	0.009	0.96	0	0.35
S1	16	7703358	176.8	0.276	0.44	15	0.25
S1	31	75074149	1723.5	2.693	0.33	35	0.25
S1	72	6266709	143.9	0.225	0.09	30	0.25
S1	104	3246876	74.5	0.116	0.14	60	0.25
S1	CmD	37289584	856.1	1.338	0.44	15	0.25
S1	Le	2310794	53.0	0.083	0.09	30	0.25
S1	Lh	17886849	410.6	0.642	0.14	60	0.25
S1	Rr	1638630	37.6	0.059	0.01	50	0.25
Total				**5.44**	**sq. miles**		
S2	8	2982573	68.5	0.107	0.96	0	0.35
S2	31	114298160	2623.9	4.100	0.33	35	0.25
S2	41	815316	18.7	0.029	0.17	0	0.25
S2	104	4606027	105.7	0.165	0.14	60	0.25
Total				**4.40**	**sq. miles**		
S3	8	5680899	130.4	0.204	0.96	0	0.35
S3	31	41310604	948.4	1.482	0.33	35	0.25
S3	33	1287326	29.6	0.046	0.23	0	0.35
S3	41	536246	12.3	0.019	0.17	0	0.25
S3	72	32004254	734.7	1.148	0.09	30	0.25
S3	95	9824	0.2	0.000	0.04	0	0.35
S3	104	11226860	257.7	0.403	0.14	60	0.25
Total				**3.30**	**sq. miles**		
S4	8	490890	11.3	0.018	0.96	0	0.35
S4	10	2310	0.1	0.000	0.94	0	0.35
S4	31	34477824	791.5	1.237	0.33	35	0.25
S4	72	7073849	162.4	0.254	0.09	30	0.25
S4	104	41081	0.9	0.001	0.14	60	0.25
S4	109	23705770	544.2	0.850	0.35	35	0.25
Total				**2.36**	**sq. miles**		
S5	8	234291	5.4	0.008	0.96	0	0.35
S5	10	1892867	43.5	0.068	0.94	0	0.35
S5	31	246625	5.7	0.009	0.33	35	0.25
S5	51	167175	3.8	0.006	0.24	0	0.15
S5	52	314449	7.2	0.011	0.16	20	0.25
S5	72	612789	14.1	0.022	0.09	30	0.25
S5	104	3513781	80.7	0.126	0.14	60	0.25
S5	108	5223664	119.9	0.187	0.31	30	0.25
S5	109	17880329	410.5	0.641	0.35	35	0.25
Total				**1.08**	**sq. miles**		

Table E.4
Summary of soils characteristics for each subbasin

Subbasin ID	Map Unit	Map Unit Area sq. feet	acres	sq. miles	XKSAT in/hr	RTIMP %	IA inches
S6	31	1120262	25.7	0.040	0.33	35	0.25
S6	72	3947035	90.6	0.142	0.09	30	0.25
S6	104	24337809	558.7	0.873	0.14	60	0.25
Total				**1.05 sq. miles**			
S7	10	2355099	54.1	0.084	0.94	0	0.35
S7	21	1445891	33.2	0.052	0.38	0	0.35
S7	48	517173	11.9	0.019	0.06	0	0.15
S7	51	7452413	171.1	0.267	0.24	0	0.15
S7	52	7998080	183.6	0.287	0.16	20	0.25
S7	68	734181	16.9	0.026	0.63	0	0.35
S7	70	667494	15.3	0.024	0.36	0	0.25
S7	103	1696371	38.9	0.061	0.1	65	0.25
S7	104	4007379	92.0	0.144	0.14	60	0.25
S7	108	4050055	93.0	0.145	0.31	30	0.25
S7	109	1992854	45.7	0.071	0.35	35	0.25
Total				**1.18 sq. miles**			
S8	10	1296245	29.8	0.046	0.94	0	0.35
S8	21	1091	0.0	0.000	0.38	0	0.35
S8	51	13498146	309.9	0.484	0.24	0	0.15
S8	52	2600653	59.7	0.093	0.16	20	0.25
S8	72	27859	0.6	0.001	0.09	30	0.25
S8	93	488079	11.2	0.018	0.33	0	0.25
S8	104	8797962	202.0	0.316	0.14	60	0.25
Total				**0.96 sq. miles**			
S9	66	1176112	27.0	0.042	0.23	0	0.35
S9	72	5461315	125.4	0.196	0.09	30	0.25
S9	93	891561	20.5	0.032	0.33	0	0.25
S9	95	3446493	79.1	0.124	0.04	0	0.35
S9	104	12067847	277.0	0.433	0.14	60	0.25
S9	10	18829	0.4	0.001	0.94	0	0.35
Total				**0.83 sq. miles**			
S10	51	2086472	47.9	0.075	0.24	0	0.15
S10	52	90683	2.1	0.003	0.16	20	0.25
S10	72	4631716	106.3	0.166	0.09	30	0.25
S10	93	128902	3.0	0.005	0.33	0	0.25
S10	95	235861	5.4	0.008	0.04	0	0.35
S10	103	698421	16.0	0.025	0.1	65	0.25
S10	104	4488989	103.1	0.161	0.14	60	0.25
Total				**0.44 sq. miles**			

Table E.4
Summary of soils characteristics for each subbasin

Subbasin ID	Map Unit	Map Unit Area sq. feet	acres	sq. miles	XKSAT in/hr	RTIMP %	IA inches
S11	10	3018665	69.3	0.108	0.94	0	0.35
S11	45	75853	1.7	0.003	0.03	0	0.25
S11	51	39589624	908.9	1.420	0.24	0	0.15
S11	52	7907624	181.5	0.284	0.16	20	0.25
S11	103	150767	3.5	0.005	0.1	65	0.25
Total				**1.82**	**sq. miles**		

Total Watershed Area = 22.87 sq. miles

Table E.5
Rainfall loss characteristics for each land use

Land Use ID (1)	Description (2)	IA inches (3)	RTIMP % (4)	Effective Vegetation Cover[2] % (5)
C1	Commercial - light	0.10	80	15
C2	Commercial - general	0.10	80	15
LDR	Low density residential	0.30	15	43
LPC	Golf course	0.20	0	80
MDR	Medium density residential	0.25	30	35
NHS	Hillslopes, Sonoran Desert[1]	0	0	0
VLDR	Very low density residential	0.30	5	29

Notes:

1. The NHS land use classification is representative of all natural conditions in the watershed. Rainfall loss parameters for these areas are accounted for under the soil map units.
2. Effective vegetation cover is the average vegetation cover for the land use area, including the impervious area.

Table E.6
Summary of land use characteristics for each subbasin

Subbasin ID	Land Use ID	Land Use Area sq. feet	Land Use Area acres	Land Use Area sq. miles	IA inches	Base RTIMP %	Vegetation Cover %
S1	NHS	151615585	3480.6	5.438	0.00	0	0
Total				**5.44 sq. miles**			
S2	C1	392040	9.0	0.014	0.10	80	15
S2	LDR	45040082	1034.0	1.616	0.30	15	43
S2	LPC	871200	20.0	0.031	0.20	0	80
S2	NHS	76398754	1753.9	2.740	0.00	0	0
Total				**4.40 sq. miles**			
S3	C1	5096520	117.0	0.183	0.10	80	15
S3	LDR	30468577	699.5	1.093	0.30	15	43
S3	LPC	435600	10.0	0.016	0.20	0	80
S3	MDR	5632377	129.3	0.202	0.25	30	35
S3	NHS	5238765	120.3	0.188	0.00	0	0
S3	VLDR	45184173	1037.3	1.621	0.30	5	29
Total				**3.30 sq. miles**			
S4	C1	1698840	39.0	0.061	0.10	80	15
S4	LDR	46936131	1077.5	1.684	0.30	15	43
S4	MDR	16759739	384.8	0.601	0.25	30	35
S4	VLDR	397013	9.1	0.014	0.30	5	29
Total				**2.36 sq. miles**			
S5	C1	217800	5.0	0.008	0.10	80	15
S5	LDR	17447520	400.5	0.626	0.30	15	43
S5	MDR	6412964	147.2	0.230	0.25	30	35
S5	NHS	3846894	88.3	0.138	0.00	0	0
S5	VLDR	2160791	49.6	0.078	0.30	5	29
Total				**1.08 sq. miles**			
S6	LDR	6382284	146.5	0.229	0.30	15	43
S6	NHS	4428854	101.7	0.159	0.00	0	0
S6	VLDR	18593968	426.9	0.667	0.30	5	29
Total				**1.05 sq. miles**			
S7	C2	1731870	39.8	0.062	0.10	80	15
S7	LDR	18456539	423.7	0.662	0.30	15	43
S7	NHS	12728581	292.2	0.457	0.00	0	0
Total				**1.18 sq. miles**			
S8	C2	3659040	84.0	0.131	0.10	80	15
S8	LDR	5274244	121.1	0.189	0.30	15	43
S8	MDR	8407188	193.0	0.302	0.25	30	35
S8	NHS	9369564	215.1	0.336	0.00	0	0
Total				**0.96 sq. miles**			

Table E.6
Summary of land use characteristics for each subbasin

Subbasin ID	Land Use ID	Land Use Area sq. feet	acres	sq. miles	IA inches	Base RTIMP %	Vegetation Cover %
S9	LDR	541886	12.4	0.019	0.30	15	43
S9	NHS	2818833	64.7	0.101	0.00	0	0
S9	VLDR	19682610	451.9	0.706	0.30	5	29
Total				**0.83**	**sq. miles**		
S10	LDR	11943390	274.2	0.428	0.30	15	43
S10	MDR	288844	6.6	0.010	0.25	30	35
S10	VLDR	147639	3.4	0.005	0.30	5	29
Total				**0.44**	**sq. miles**		
S11	LDR	1010151	23.2	0.036	0.30	15	43
S11	MDR	9575186	219.8	0.343	0.25	30	35
S11	NHS	40157196	921.9	1.440	0.00	0	0
Total				**1.82**	**sq. miles**		
Total Watershed Area =				**22.87**	**sq. miles**		

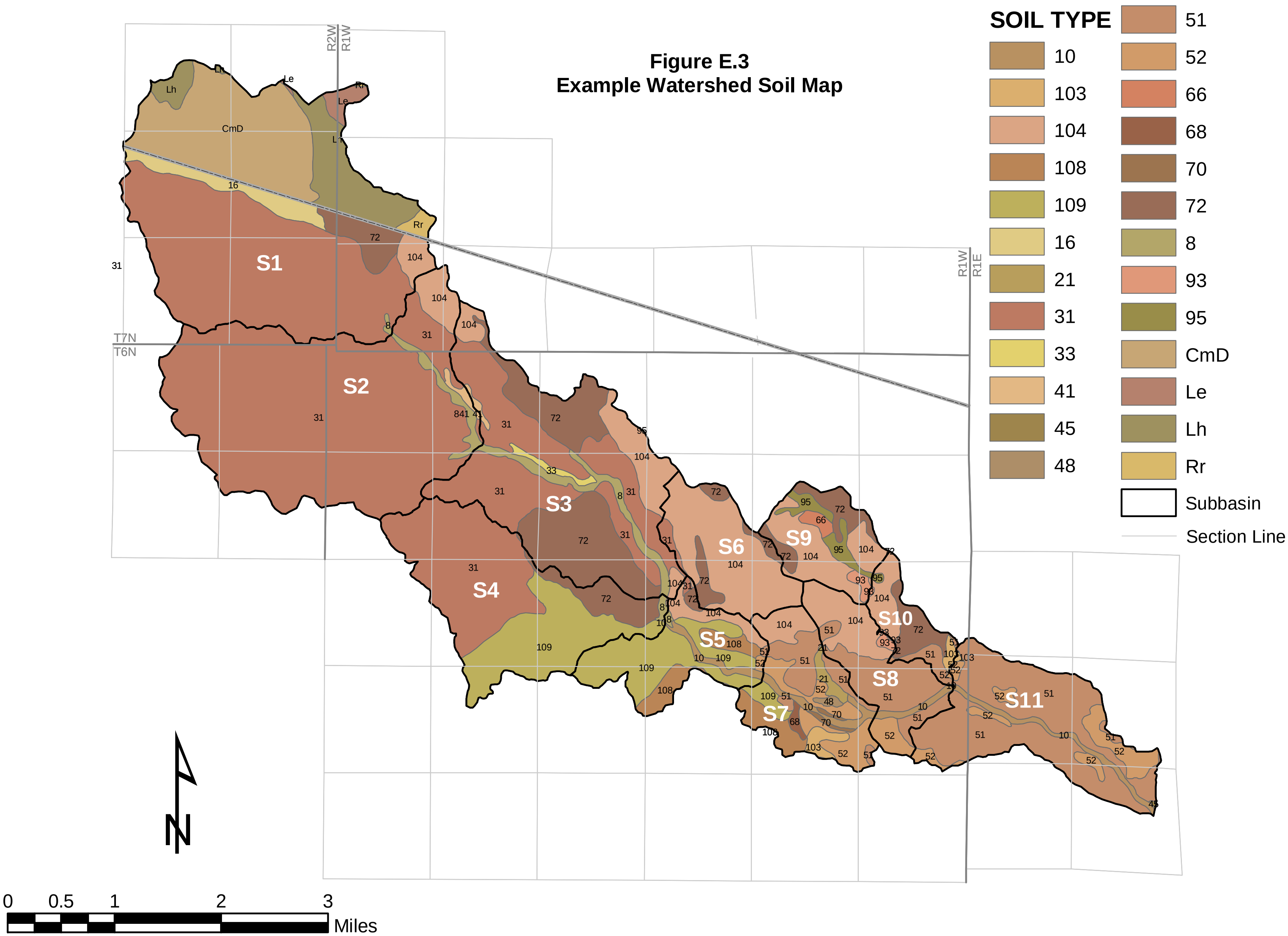
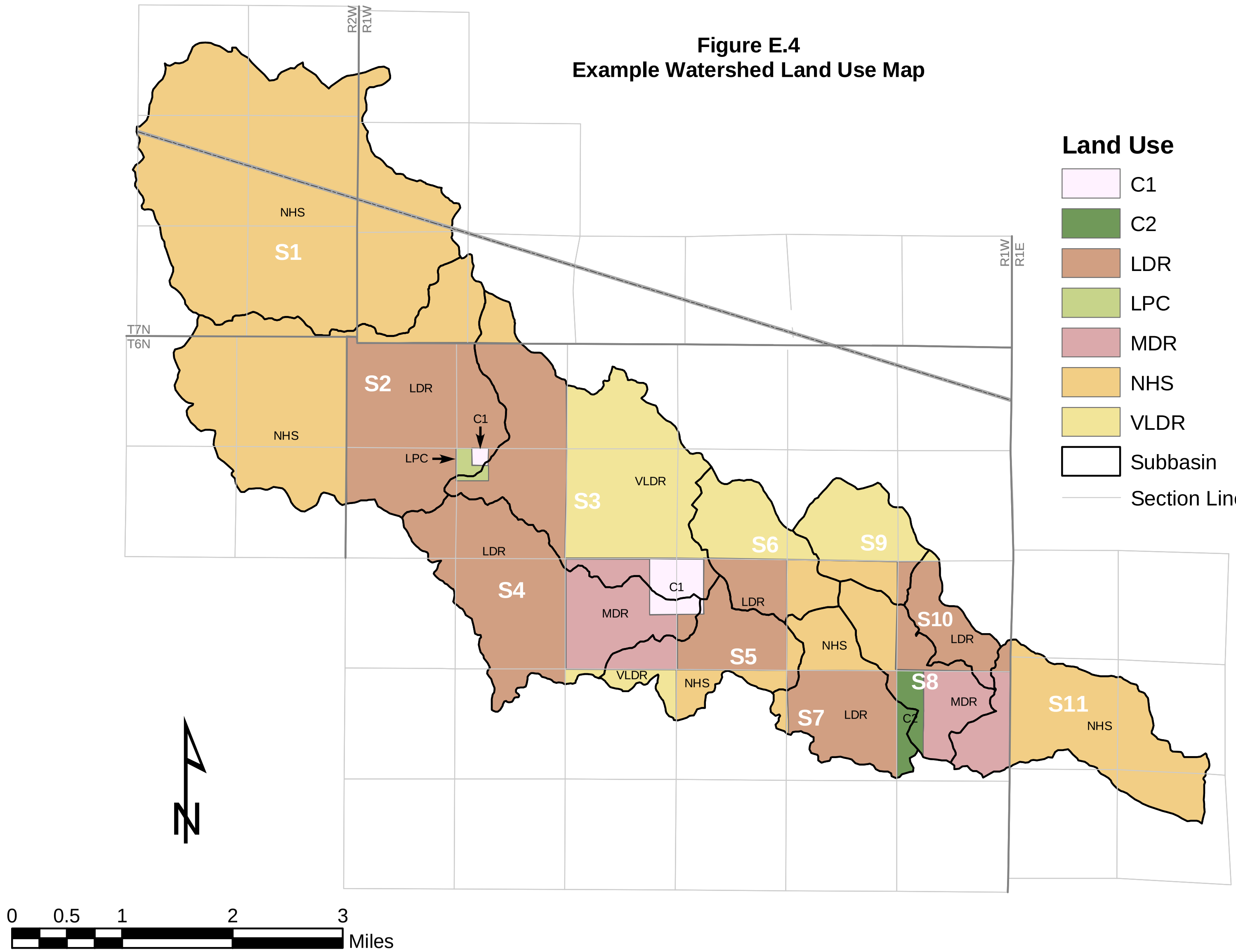


Figure E.3
Example Watershed Soil Map

Figure E.4
Example Watershed Land Use Map



9.4 UNIT HYDROGRAPH

9.4.1 Procedures for the Clark Unit Hydrograph

1. From an appropriate map of the watershed, measure drainage area (A) and the values of L and S.

2. If S is greater than 200 ft/mi, adjust the slope using Table 5.2 or Figure 5.4.

3. Using either Figure 5.5 or Table 5.3, select a resistance coefficient (K_b) for the basin or subbasin based on a resistance classification and the drainage area (in acres). For a basin or subbasin of mixed classification;

   - A representative K_b can be interpolated from Figure 5.5, or
   - An arithmetically averaged K_b can be calculated based on the area of each unique K_b present in the basin or subbasin.

4. Calculate T_c as a function of i using Equation 5.5

5. Enter the following data into an HEC-1 input file:

   - Design rainfall per the methodology and procedures in Chapter 2;
   - Basin area;
   - Rainfall loss data per the methodologies and procedures in Chapter 4; and
   - Clark unit hydrograph parameters (values set to zero).

6. Run HEC-1 with the input file from Step 5 at an output level of zero for each subbasin. Using the T_c worksheet (Appendix D, Section 1), tabulate the period of peak rainfall excess for each subbasin and compute the average intensities to a time greater than the expected T_c.

7. Construct the graph of average rainfall excess intensity vs. time and calculate T_c by iteration.

8. Calculate R using Equation 5.6.

9. Select the appropriate time-area relation for the basin or subbasin.

As an alternative to the above procedures, the DDMSW program will compute the rainfall excess directly and perform the necessary iterations to compute the T_c and R parameters.

9.4.2 Procedures for the S-Graph

1. From an appropriate map of the watershed, measure drainage area (A), L, L_{ca} and S.

2. Calculate the basin factor $\frac{LL_{ca}}{S^{0.5}}$.

3. Using the data in Appendix D, Section 2 or the tables in the Design of Small Dams or the USBR Flood Hydrology Manual, attempt to identify watersheds of the same physiographic type and similar drainage area and basin factor. Make a list of the watersheds with similar drainage areas and basin factors and tabulate the estimated value of K_n for those watersheds and the measured lag.

4. Estimate K_n for the watershed by inspection of the tabulation from Step 3.

5. Calculate the coefficient (C) and select the value of the exponent (m) corresponding to the source (Corps of Engineers or USBR) that was used to estimate K_n. If the source of K_n is unknown, then use the Corps of Engineers version of Equation 5.11.

6. Using Equation 5.11, calculate the basin lag. Compare this value to the measured lags of watersheds from Step 3.

7. Select an appropriate computational time interval (NMIN) and compute Q_{ult} using Equation 5.10.

8. Select an appropriate S-Graph and tabulate the percent Q_{ult}, percent lag and the accumulated time.

9. Transform the S-Graph into an X-duration (NMIN) unit hydrograph using linear interpolation with Δt = NMIN.

10. Adjust the “tail” region of the S-Graph by lagging that portion by Δt and subtracting the ordinates.

As an alternative to the above procedure, the DDMSW will transform the S-Graph to a unit graph automatically.

9.4.3 User Notes

9.4.3.1 Clark Unit Hydrograph

1. The Clark Unit Hydrograph procedure was developed from a database that included both urban and natural (undeveloped) desert/rangeland watersheds. The primary application of the Clark Unit Hydrograph is for urban watersheds, but it is also applicable for undeveloped desert/rangeland watersheds. In general, the Clark Unit Hydrograph is not applicable to agricultural fields or steep mountain watersheds.

2. The following limitations apply to the Clark Unit Hydrograph procedure.

    a. The recommended drainage area limit is 5 square miles with a maximum of 10 square miles.

    b. The calculated T_c should not exceed the duration of rainfall excess.

    c. The calculated T_c should not be longer than 1.5 hours.

   If a drainage basin does not meet any or all of the preceding limitations, then the following options are available:

    - Subdivide the drainage area into smaller subbasins such that all of these subbasins satisfy the limitations.
    - Use the S-Graph method, provided the drainage basin satisfies the limitations of that method.
    - Justify the use of an alternative approach.

3. Time of concentration as defined in this manual is the travel time, during the corresponding period of the most intense portion of rainfall excess, for a floodwave to travel from the hydraulically most distant point in the watershed to the point of interest. The determination of the hydraulically most distant point is made in regard to both length and slope. In other words, the hydraulically most distant point is not necessarily the longest length, but may be a shorter length with an appreciably flatter slope.

4. When calculating the T_c for a natural watershed, with slopes greater than 200 ft/mile, use Figure 5.4 to adjust the slope. The use of the adjusted slope should be considered when determining the T_c of the hydraulically most distant point.

5. T_c is a function of rainfall excess and must be recalculated for each desired frequency or design storm duration.

6. If hand calculating the T_c observe the following:

   a. Ranking of the rainfall excess values is by “nesting” of the largest values around the maximum value, not ordering from largest to smallest. Example:

Time	Excess (in)	Rank
1415	0.21	6
1420	0.28	5
1425	0.35	2
1430	0.40	1
1435	0.32	3
1440	0.33	4
1445	0.18	7

   b. The T_c calculation worksheet allows for a maximum of eight excess values to be entered, and this is sufficient in most cases. As a result, if Δt = 5 min, then T_c should be less than (8 * 5) = 40 min. For Δt = 10 min, T_c < 80 min, and so on. In no case should T_c be less than Δt for computational stability.

7. If a time-area relation is not specified in the HEC-1 model, then the HEC-1 default time-area relation is used which, in general, is not recommended for use in Maricopa County.

9.4.3.2 S-Graph

1. The recommended S-Graphs for Maricopa County (i.e. Phoenix Mountain, Phoenix Valley, Desert/Rangeland, and Agricultural) should only be applied to large natural watersheds. The Phoenix Valley S-Graph is also applicable to large urban watersheds. This is, in part, due to the fact that the original database in Arizona applied the methodology to large watersheds. As a lower limit of application a watershed area of 5 square miles can be considered.

2. K_n should be selected from the best available information. General guidance and some regional data is available from the U.S. Army Corps of Engineers (Figure 5.11). A broader range of data for watersheds in Maricopa County is provided in the USBR Flood Hydrology Manual (Cudworth, 1989). The S-Graph study (Sabol, 1987) contains lag and watershed characteristics data that are not generally contained in other publications. These sources should be consulted when selecting K_n.

3. The manual discusses two slightly different forms of the lag equation (Equation 5.11), one by the U.S. Army Corps of Engineers and one by the USBR. The form of the equation that corresponds to the source used in the selection of K_n should be used.

4. The length to the basin centroid (L_{ca}) is measured along L to a point on L that is opposite (perpendicular to) the basin centroid. L_{ca} is not measured to the centroid unless the centroid happens to lie on the flow path line (L).

5. The transformation of an S-Graph to a unit graph is a function of the selected computational time interval (NMIN). If a new NMIN is desired a new unit graph must be recalculated.

6. The slope as applied in the calculation of basin lag is not adjusted, regardless of the value.

9.4.4 Example

Compute the 6-hour unit hydrograph parameters for each subbasin shown in Figure E.1 using rainfall data and rainfall loss data from the examples in Chapters 2 and 4, respectively and the following data

Subbasin ID	Drainage Area sq. miles	Flow Path Length miles	Flow Path L_{ca} [1] miles	Flow Path Slope [2] ft/mi
S1	5.44	4.59	2.30	255
S2	4.40	4.10	1.95	207
S3	3.30	3.90	1.71	307
S4	2.36	3.40	1.36	200
S5	1.08	2.28	0.95	149
S6	1.05	2.06	0.98	136
S7	1.18	1.74	0.72	158
S8	0.96	2.36	1.40	199
S9	0.83	1.32	0.75	99
S10	0.44	1.70	0.79	228
S11	1.82	2.98	1.34	128

Notes:

1. L_{ca} is given for each subbasin regardless of the unit hydrograph method
2. Average slope, not the adjusted slope

Solution

1. Select the appropriate unit hydrograph method for each subbasin

   - For subbasin S1, the Phoenix Mountain S-graph is selected. For all other subbasins, the Clark unit hydrograph is selected.

2. Develop the unit hydrograph for Subbasin S1

a. Compute the basin factor $\left(\frac{LL_{ca}}{S^{0.5}}\right)$

$$\frac{LL_{ca}}{S^{0.5}} = \frac{(4.59)(2.30)}{255^{0.5}} = 0.66$$

b. Select a value for K_n

From Appendix D, Section 2 for mountain and foothill watersheds, the Santa Anita Creek and Medicine Bow River watersheds were found to have similar physical characteristics to Subbasin S1. The K_n values for those watersheds are 0.053 and 0.0534, respectively with lag times of 1.10 and 0.89 hours, respectively. Comparison of the K_n values for these two watersheds to the general values of K_n for the Phoenix Mountain S-Graph, provided in Table 5.6, indicate that a value for K_n of 0.053 is appropriate.

$K_n = 0.053$

c. Compute the lag time using equation 5.11

The source of the K_n values for the two similar watersheds is unknown, therefore use the Corps of Engineers version of the lag equation.

$C = 24\,K_n = (24)(0.053) = 1.272$

$m = 0.38$

$$Lag = C\left(\frac{LL_{ca}}{S^{0.5}}\right)^{m} = 1.272\,(0.66)^{0.38} = 1.09 \text{ hours}$$

The lag of 1.09 hours compares favorably to the lag times of the similar watersheds used for the selection of K_n.

d. Compute Q_{ult} using equation 5.10

$$Q_{ult} = \frac{645.33A}{\Delta t}$$

$\Delta t = 0.15\ lag = (0.15)(1.09) = 0.164$ hours, therefore use $\Delta t = 10$ minutes

$$Q_{ult} = \frac{(645.33)(5.44)}{(10/60)} = 21{,}064 \text{ cfs}$$

e. Compute the discharge and lag corresponding to the values for percent Q_{ult} and percent Lag in Table 5.5

Percent Q_{ult} (1)	Discharge cfs (2)	Percent Lag (3)	Lag hours (4)	Percent Q_{ult} (1)	Discharge cfs (2)	Percent Lag (3)	Lag hours (4)
0	0	0.0	0.00	52	10953	103.4	1.13
2	421	23.0	0.25	54	11375	107.0	1.17
4	843	31.0	0.34	56	11796	110.8	1.21
6	1264	37.0	0.40	58	12217	114.7	1.25
8	1685	42.0	0.46	60	12638	118.7	1.29
10	2106	46.0	0.50	62	13060	122.9	1.34
12	2528	49.8	0.54	64	13481	127.3	1.39
14	2949	53.4	0.58	66	13902	131.9	1.44
16	3370	56.8	0.62	68	14324	136.7	1.49
18	3792	60.0	0.65	70	14745	141.7	1.54
20	4213	63.1	0.69	72	15166	147.1	1.60
22	4634	66.1	0.72	74	15587	152.8	1.67
24	5055	69.0	0.75	76	16009	158.8	1.73
26	5477	71.8	0.78	78	16430	165.5	1.80
28	5898	74.4	0.81	80	16851	172.9	1.88
30	6319	76.8	0.84	82	17272	181.6	1.98
32	6740	79.1	0.86	84	17694	191.0	2.08
34	7162	81.2	0.89	86	18115	201.0	2.19
36	7583	83.2	0.91	88	18536	212.0	2.31
38	8004	85.1	0.93	90	18958	226.0	2.46
40	8426	86.8	0.95	92	19379	244.0	2.66
42	8847	88.8	0.97	94	19800	265.0	2.89
44	9268	91.0	0.99	96	20221	295.0	3.22
46	9689	93.8	1.02	98	20643	342.0	3.73
48	10111	96.8	1.06	100	21064	462.0	5.04
50	10532	100.0	1.09				

Notes:

(1) = From Table 5.5
(2) = (1) * Q_{ult}
(3) = From Table 5.5
(4) = (3) * Lag

f. Transform the S-graph into a 10-minute Unit Hydrograph

Time hours (1)	Q_1 cfs (2)	Q2 cfs (3)	Q_{unit} cfs (4)
0.000	0	0	0
0.167	280	0	280
0.333	820	280	540
0.500	2093	820	1272
0.667	3949	2093	1857
0.833	6258	3949	2309
1.000	9380	6258	3122
1.167	11378	9380	1998
1.333	13002	11378	1624
1.500	14401	13002	1399
1.667	15595	14401	1194
1.833	16583	15595	989
2.000	17357	16583	774
2.167	18021	17357	664
2.333	18599	18021	577
2.500	19036	18599	438
2.667	19392	19036	356
2.833	19699	19392	307
3.000	19944	19699	245
3.167	20159	19944	215
3.333	20318	20159	160
3.500	20455	20318	137
3.667	20592	20455	137
3.833	20677	20592	84
4.000	20730	20677	54
4.167	20784	20730	54
4.333	20838	20784	54
4.500	20891	20838	54
4.667	20945	20891	54
4.833	20999	20945	54
5.000	21052	20999	54
5.167	21064	21052	12
5.333	21064	21064	0

Notes:
- (2) = Linear interpolation from previous table, column 2
- (3) = (2) lagged 10-minutes
- (4) = (2) - (3)

3. Calculate the Clark unit hydrograph parameters for subbasins S2 through S11

a. Using Table 5.2 or Figure 5.4, determine the adjusted slope for subbasins S2, S3 and S10 (subbasins with average slopes greater than 200 ft/mi).

	Slope	
Subbasin ID	Average ft/mi	Adjusted ft/mi
S2	207	206
S3	307	266
S10	228	224

b. Compute the Resistance Coefficient (K_b) using Table 5.3. For natural areas K_b type C is selected and K_b type A is selected for the urban areas.

- Subbasin S2

$$K_b = m \log A + b$$

$$K_b^N = -0.025 \log(1063.0) + 0.15 = 0.074$$

$$K_b^D = -0.00625 \log(1753.9) + 0.04 = 0.020$$

$$K_b^W = (0.62)(0.074) + (0.38)(0.020) = 0.053$$

Using the above procedures, the K_b for each of the Clark subbasins is as follows:

Subbasin ID	Drainage Area acres	Percent Natural	Percent Developed	K_b Natural	K_b Developed	K_b Weighted
S2	2816.9	62	38	0.074	0.020	0.054
S3	2113.3	6	94	0.098	0.019	0.024
S4	1510.4	0	100	---	0.020	0.020
S5	690.7	13	87	0.101	0.023	0.033
S6	675.1	15	85	0.100	0.023	0.034
S7	755.7	39	61	0.088	0.023	0.048
S8	613.2	35	65	0.092	0.024	0.048
S9	529.0	12	88	0.105	0.023	0.033
S10	284.2	0	100	---	0.025	0.025
S11	1164.9	79	21	0.076	0.025	0.065

c. Compute Time of Concentration (T_c) as a function of Intensity (i) using equation 5.5

- Subbasin S2

$$T_c = 11.4\, L^{0.5} K_b^{0.52} S^{-0.31} i^{-0.38}$$

$$T_c = (11.4)(4.10)^{0.5}(0.054)^{0.52}(206)^{-0.31} i^{-0.38}$$

$$T_c = 0.97\, i^{-0.38}$$

Using the above procedure, T_c as a function of i for each of the Clark subbasins is as follows:

Subbasin ID	Length miles	K_b	Adjusted Slope ft/mi	T_c as a Function of i
S2	4.10	0.054	206	0.97
S3	3.90	0.024	266	0.57
S4	3.40	0.020	200	0.53
S5	2.28	0.033	149	0.62
S6	2.06	0.034	136	0.62
S7	1.74	0.048	158	0.65
S8	2.36	0.048	199	0.70
S9	1.32	0.033	99	0.54
S10	1.70	0.025	224	0.41
S11	2.98	0.065	128	1.06

d. Develop a dummy HEC-1 model using the 6-hour rainfall data and the procedures from the Chapter 2 example and the rainfall loss parameters from the Chapter 4 example to compute rainfall excess.

Note: For the purpose of this example, only the dummy model for subbasin S2 is provided

e. Using the peak rainfall excess from the model output, construct a graph of average excess intensity vs time using the T_c worksheet provided in Appendix D, Section 1 and compute T_c and R.

Note: For the purpose of this example, only the T_c worksheet for subbasin S2 is provided.

```
1***************************************                      ***************************************
 *                                     *                      *                                     *
 *   FLOOD HYDROGRAPH PACKAGE  (HEC-1)  *                      *   U.S. ARMY CORPS OF ENGINEERS      *
 *             JUN   1998               *                      *   HYDROLOGIC ENGINEERING CENTER     *
 *           VERSION 4.1                *                      *        609 SECOND STREET            *
 *                                     *                      *      DAVIS, CALIFORNIA 95616        *
 *  RUN DATE   12APR02  TIME  16:59:03  *                      *          (916) 756-1104             *
 *                                     *                      *                                     *
***************************************                      ***************************************


                    X     X  XXXXXXX   XXXXX         X
                    X     X  X        X     X       XX
                    X     X  X        X              X
                    XXXXXXX  XXXX     X       XXXXX  X
                    X     X  X        X              X
                    X     X  X        X     X        X
                    X     X  XXXXXXX   XXXXX        XXX


  THIS PROGRAM REPLACES ALL PREVIOUS VERSIONS OF HEC-1 KNOWN AS HEC1 (JAN 73), HEC1GS, HEC1DB, AND HEC1KW.

  THE DEFINITIONS OF VARIABLES -RTIMP- AND -RTIOR- HAVE CHANGED FROM THOSE USED WITH THE 1973-STYLE INPUT STRUCTURE.
  THE DEFINITION OF -AMSKK- ON RM-CARD WAS CHANGED WITH REVISIONS DATED 28 SEP 81. THIS IS THE FORTRAN77 VERSION
  NEW OPTIONS: DAMBREAK OUTFLOW SUBMERGENCE , SINGLE EVENT DAMAGE CALCULATION, DSS:WRITE STAGE FREQUENCY,
  DSS:READ TIME SERIES AT DESIRED CALCULATION INTERVAL   LOSS RATE:GREEN AND AMPT INFILTRATION
  KINEMATIC WAVE: NEW FINITE DIFFERENCE ALGORITHM

1                                         HEC-1 INPUT                                          PAGE  1

          ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10

          ID  Flood Control District of Maricopa County
          ID  Chapter 5 Example - Subbasin S1
          ID
          ID  Dummy HEC-1 model to compute the rainfall excess for each Clark subbasin.
          ID  The rainfall excess is used to determine the Tc.
          ID
          ID  Rainfall is for the 100-year, 6-hour storm.  The rainfall depths,
          ID  distribution and aerial reduction factors are taken from the Chapter 2
          ID  example.
          ID
          ID  Rainfall losses are estimated using the Green and Ampt methodology.  The
          ID  values coded on the LG record set are taken from the Chapter 4 example.
          IT     10                    300
          IO      1
          *

          KK     S2
          KM  Subbasin     S2
          KM
          KM     The Clark Unit Hydrograph is used for this basin.
          KM         The Natural time-area relation is used for this basin.
          KM
          KM     Time of Concentration for this subbasin is based on the following:
          KM         6-Hour Rainfall, Pattern No.  2.26
          KM         An rainfall areal reduction factor of 0.964
          KM
          KM      EXCESS RAINFALL VALUES EXCEEDED IN 5-MINUTE INTERVALS
          KM    5  10  15  20  25  30  35  40  45  50  55  60  65  70  75  80  85  90
          KM  .21 .20 .20 .16 .15 .14 .11 .11 .11 .04 .04 .04 .03 .03 .03 .02 .02 .02
          KM
          BA  4.400
          IN     15
          KM  Rainfall depth of  3.40 was spacially reduced as shown by the PB record.
          KM  An areal reduction coefficient of 0.964 is used.
          PB   3.28
          KM  The following PC record used a 6-hour rainfall with Pattern No.  2.26
          PC  0.000   0.011   0.017   0.026   0.038   0.047   0.057   0.067   0.077   0.087
          PC  0.099   0.113   0.134   0.178   0.265   0.456   0.688   0.826   0.892   0.931
          PC  0.949   0.962   0.974   0.988   1.000
          LG   0.27    0.31    4.34    0.41   39.00
          UC
          *
```

```
            40          ZZ
1***************************************                           ***************************************
 *                                     *                           *                                     *
 *   FLOOD HYDROGRAPH PACKAGE  (HEC-1)  *                           *    U.S. ARMY CORPS OF ENGINEERS     *
 *              JUN   1998              *                           *    HYDROLOGIC ENGINEERING CENTER    *
 *            VERSION 4.1               *                           *         609 SECOND STREET           *
 *                                     *                           *      DAVIS, CALIFORNIA 95616        *
 *  RUN DATE   12APR02  TIME  16:59:03  *                           *          (916) 756-1104             *
 *                                     *                           *                                     *
***************************************                            ***************************************


                         Flood Control District of Maricopa County
                         Chapter 5 Example - Subbasin S1

                         Dummy HEC-1 model to compute the rainfall excess for each Clark subbasin.
                         The rainfall excess is used to determine the Tc.

                         Rainfall is for the 100-year, 6-hour storm.  The rainfall depths,
                         distribution and aerial reduction factors are taken from the Chapter 2
                         example.

                         Rainfall losses are estimated using the Green and Ampt methodology.  The
                         values coded on the LG record set are taken from the Chapter 4 example.

  14 IO          OUTPUT CONTROL VARIABLES
                       IPRNT          1  PRINT CONTROL
                       IPLOT          0  PLOT CONTROL
                       QSCAL          0.  HYDROGRAPH PLOT SCALE

    IT           HYDROGRAPH TIME DATA
                        NMIN         10  MINUTES IN COMPUTATION INTERVAL
                       IDATE      1    0  STARTING DATE
                       ITIME       0000  STARTING TIME
                          NQ        300  NUMBER OF HYDROGRAPH ORDINATES
                      NDDATE      3    0  ENDING DATE
                      NDTIME       0150  ENDING TIME
                       ICENT         19  CENTURY MARK

                    COMPUTATION INTERVAL    0.17 HOURS
                         TOTAL TIME BASE   49.83 HOURS

        ENGLISH UNITS
             DRAINAGE AREA          SQUARE MILES
             PRECIPITATION DEPTH    INCHES
             LENGTH, ELEVATION      FEET
             FLOW                   CUBIC FEET PER SECOND
             STORAGE VOLUME         ACRE-FEET
             SURFACE AREA           ACRES
             TEMPERATURE            DEGREES FAHRENHEIT


*** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** ***

            **************
            *            *
  15 KK     *       S2   *
            *            *
            **************
                         Subbasin     S2

                            The Clark Unit Hydrograph is used for this basin.
                                The Natural time-area relation is used for this basin.

                            Time of Concentration for this subbasin is based on the following:
                                6-Hour Rainfall, Pattern No.  2.26
                                An rainfall areal reduction factor of 0.964

                             EXCESS RAINFALL VALUES EXCEEDED IN 5-MINUTE INTERVALS
                           5  10  15  20  25  30  35  40  45  50  55  60  65  70  75  80  85  90
                         .21 .20 .20 .16 .15 .14 .11 .11 .11 .04 .04 .04 .03 .03 .03 .02 .02 .02

                         Rainfall depth of  3.40 was spacially reduced as shown by the PB record.
                         An areal reduction coefficient of 0.964 is used.
                         The following PC record used a 6-hour rainfall with Pattern No.  2.26
```

```
 30 IN          TIME DATA FOR INPUT TIME SERIES
                     JXMIN        15  TIME INTERVAL IN MINUTES
                    JXDATE      1  0  STARTING DATE
                    JXTIME         0  STARTING TIME

              SUBBASIN RUNOFF DATA

 29 BA          SUBBASIN CHARACTERISTICS
                     TAREA      4.40  SUBBASIN AREA

                PRECIPITATION DATA

 34 PB               STORM      3.28  BASIN TOTAL PRECIPITATION

 34 PI            INCREMENTAL PRECIPITATION PATTERN
                   0.01     0.01     0.00     0.01     0.01     0.01     0.01     0.01     0.01     0.01
                   0.01     0.01     0.01     0.01     0.01     0.01     0.01     0.01     0.03     0.04
                   0.06     0.13     0.14     0.15     0.09     0.07     0.04     0.03     0.02     0.01
                   0.01     0.01     0.01     0.01     0.01     0.01

 38 LG          GREEN AND AMPT LOSS RATE
                     STRTL      0.27  STARTING LOSS
                       DTH      0.31  MOISTURE DEFICIT
                      PSIF      4.34  WETTING FRONT SUCTION
                     XKSAT      0.41  HYDRAULIC CONDUCTIVITY
                     RTIMP     39.00  PERCENT IMPERVIOUS AREA

 39 UC          CLARK UNITGRAPH
                        TC      0.00  TIME OF CONCENTRATION
                         R      0.00  STORAGE COEFFICIENT

                SYNTHETIC ACCUMULATED-AREA VS. TIME CURVE WILL BE USED

                                                        ***

TC INCREASED TO DELTA T OF  0.17 HR
R INCREASED TO MINIMUM OF 0.5

                                                   UNIT HYDROGRAPH PARAMETERS
                                             CLARK   TC=  0.17 HR,       R=  0.08 HR
                                            SNYDER   TP=  0.13 HR,      CP=  0.50

                                                        UNIT HYDROGRAPH
                                                   2 END-OF-PERIOD ORDINATES
              8518.     8518.

*************************************************************************************************************************************

                                                 HYDROGRAPH AT STATION        S2

*************************************************************************************************************************************
                                                             *
      DA MON HRMN  ORD     RAIN    LOSS  EXCESS     COMP Q   *        DA MON HRMN  ORD     RAIN    LOSS  EXCESS     COMP Q
                                                             *
       1    0000    1     0.00    0.00    0.00         0.    *         2    0100  151     0.00    0.00    0.00         0.
       1    0010    2     0.02    0.01    0.01        80.    *         2    0110  152     0.00    0.00    0.00         0.
       1    0020    3     0.02    0.01    0.01       142.    *         2    0120  153     0.00    0.00    0.00         0.
       1    0030    4     0.01    0.01    0.01       105.    *         2    0130  154     0.00    0.00    0.00         0.
       1    0040    5     0.02    0.01    0.01       109.    *         2    0140  155     0.00    0.00    0.00         0.
       1    0050    6     0.02    0.01    0.01       142.    *         2    0150  156     0.00    0.00    0.00         0.
       1    0100    7     0.03    0.02    0.01       163.    *         2    0200  157     0.00    0.00    0.00         0.
       1    0110    8     0.02    0.01    0.01       153.    *         2    0210  158     0.00    0.00    0.00         0.
       1    0120    9     0.02    0.01    0.01       134.    *         2    0220  159     0.00    0.00    0.00         0.
       1    0130   10     0.02    0.01    0.01       142.    *         2    0230  160     0.00    0.00    0.00         0.
       1    0140   11     0.02    0.01    0.01       145.    *         2    0240  161     0.00    0.00    0.00         0.
       1    0150   12     0.02    0.01    0.01       145.    *         2    0250  162     0.00    0.00    0.00         0.
       1    0200   13     0.02    0.01    0.01       145.    *         2    0300  163     0.00    0.00    0.00         0.
       1    0210   14     0.02    0.01    0.01       145.    *         2    0310  164     0.00    0.00    0.00         0.
       1    0220   15     0.02    0.01    0.01       153.    *         2    0320  165     0.00    0.00    0.00         0.
       1    0230   16     0.03    0.02    0.01       167.    *         2    0330  166     0.00    0.00    0.00         0.
       1    0240   17     0.03    0.02    0.01       189.    *         2    0340  167     0.00    0.00    0.00         0.
       1    0250   18     0.04    0.02    0.01       229.    *         2    0350  168     0.00    0.00    0.00         0.
       1    0300   19     0.05    0.03    0.02       280.    *         2    0400  169     0.00    0.00    0.00         0.
       1    0310   20     0.10    0.06    0.04       472.    *         2    0410  170     0.00    0.00    0.00         0.
       1    0320   21     0.14    0.09    0.06       795.    *         2    0420  171     0.00    0.00    0.00         0.
       1    0330   22     0.19    0.12    0.07      1108.    *         2    0430  172     0.00    0.00    0.00         0.
       1    0340   23     0.42    0.12    0.30      3150.    *         2    0440  173     0.00    0.00    0.00         0.
       1    0350   24     0.46    0.11    0.36      5562.    *         2    0450  174     0.00    0.00    0.00         0.
       1    0400   25     0.51    0.10    0.41      6555.    *         2    0500  175     0.00    0.00    0.00         0.
```

1	0410	26	0.30	0.09	0.21	5328.	*	2	0510	176	0.00	0.00	0.00	0.
1	0420	27	0.22	0.08	0.14	3006.	*	2	0520	177	0.00	0.00	0.00	0.
1	0430	28	0.14	0.08	0.06	1739.	*	2	0530	178	0.00	0.00	0.00	0.
1	0440	29	0.09	0.05	0.03	834.	*	2	0540	179	0.00	0.00	0.00	0.
1	0450	30	0.06	0.04	0.02	490.	*	2	0550	180	0.00	0.00	0.00	0.
1	0500	31	0.04	0.02	0.02	338.	*	2	0600	181	0.00	0.00	0.00	0.
1	0510	32	0.03	0.02	0.01	225.	*	2	0610	182	0.00	0.00	0.00	0.
1	0520	33	0.03	0.02	0.01	185.	*	2	0620	183	0.00	0.00	0.00	0.
1	0530	34	0.03	0.02	0.01	178.	*	2	0630	184	0.00	0.00	0.00	0.
1	0540	35	0.03	0.02	0.01	189.	*	2	0640	185	0.00	0.00	0.00	0.
1	0550	36	0.03	0.02	0.01	196.	*	2	0650	186	0.00	0.00	0.00	0.
1	0600	37	0.03	0.02	0.01	182.	*	2	0700	187	0.00	0.00	0.00	0.
1	0610	38	0.00	0.00	0.00	87.	*	2	0710	188	0.00	0.00	0.00	0.
1	0620	39	0.00	0.00	0.00	0.	*	2	0720	189	0.00	0.00	0.00	0.
1	0630	40	0.00	0.00	0.00	0.	*	2	0730	190	0.00	0.00	0.00	0.
1	0640	41	0.00	0.00	0.00	0.	*	2	0740	191	0.00	0.00	0.00	0.
1	0650	42	0.00	0.00	0.00	0.	*	2	0750	192	0.00	0.00	0.00	0.
1	0700	43	0.00	0.00	0.00	0.	*	2	0800	193	0.00	0.00	0.00	0.
1	0710	44	0.00	0.00	0.00	0.	*	2	0810	194	0.00	0.00	0.00	0.
1	0720	45	0.00	0.00	0.00	0.	*	2	0820	195	0.00	0.00	0.00	0.
1	0730	46	0.00	0.00	0.00	0.	*	2	0830	196	0.00	0.00	0.00	0.
1	0740	47	0.00	0.00	0.00	0.	*	2	0840	197	0.00	0.00	0.00	0.
1	0750	48	0.00	0.00	0.00	0.	*	2	0850	198	0.00	0.00	0.00	0.
1	0800	49	0.00	0.00	0.00	0.	*	2	0900	199	0.00	0.00	0.00	0.
1	0810	50	0.00	0.00	0.00	0.	*	2	0910	200	0.00	0.00	0.00	0.
1	0820	51	0.00	0.00	0.00	0.	*	2	0920	201	0.00	0.00	0.00	0.
1	0830	52	0.00	0.00	0.00	0.	*	2	0930	202	0.00	0.00	0.00	0.
1	0840	53	0.00	0.00	0.00	0.	*	2	0940	203	0.00	0.00	0.00	0.
1	0850	54	0.00	0.00	0.00	0.	*	2	0950	204	0.00	0.00	0.00	0.
1	0900	55	0.00	0.00	0.00	0.	*	2	1000	205	0.00	0.00	0.00	0.
1	0910	56	0.00	0.00	0.00	0.	*	2	1010	206	0.00	0.00	0.00	0.
1	0920	57	0.00	0.00	0.00	0.	*	2	1020	207	0.00	0.00	0.00	0.
1	0930	58	0.00	0.00	0.00	0.	*	2	1030	208	0.00	0.00	0.00	0.
1	0940	59	0.00	0.00	0.00	0.	*	2	1040	209	0.00	0.00	0.00	0.
1	0950	60	0.00	0.00	0.00	0.	*	2	1050	210	0.00	0.00	0.00	0.
1	1000	61	0.00	0.00	0.00	0.	*	2	1100	211	0.00	0.00	0.00	0.
1	1010	62	0.00	0.00	0.00	0.	*	2	1110	212	0.00	0.00	0.00	0.
1	1020	63	0.00	0.00	0.00	0.	*	2	1120	213	0.00	0.00	0.00	0.
1	1030	64	0.00	0.00	0.00	0.	*	2	1130	214	0.00	0.00	0.00	0.
1	1040	65	0.00	0.00	0.00	0.	*	2	1140	215	0.00	0.00	0.00	0.
1	1050	66	0.00	0.00	0.00	0.	*	2	1150	216	0.00	0.00	0.00	0.
1	1100	67	0.00	0.00	0.00	0.	*	2	1200	217	0.00	0.00	0.00	0.
1	1110	68	0.00	0.00	0.00	0.	*	2	1210	218	0.00	0.00	0.00	0.
1	1120	69	0.00	0.00	0.00	0.	*	2	1220	219	0.00	0.00	0.00	0.
1	1130	70	0.00	0.00	0.00	0.	*	2	1230	220	0.00	0.00	0.00	0.
1	1140	71	0.00	0.00	0.00	0.	*	2	1240	221	0.00	0.00	0.00	0.
1	1150	72	0.00	0.00	0.00	0.	*	2	1250	222	0.00	0.00	0.00	0.
1	1200	73	0.00	0.00	0.00	0.	*	2	1300	223	0.00	0.00	0.00	0.
1	1210	74	0.00	0.00	0.00	0.	*	2	1310	224	0.00	0.00	0.00	0.
1	1220	75	0.00	0.00	0.00	0.	*	2	1320	225	0.00	0.00	0.00	0.
1	1230	76	0.00	0.00	0.00	0.	*	2	1330	226	0.00	0.00	0.00	0.
1	1240	77	0.00	0.00	0.00	0.	*	2	1340	227	0.00	0.00	0.00	0.
1	1250	78	0.00	0.00	0.00	0.	*	2	1350	228	0.00	0.00	0.00	0.
1	1300	79	0.00	0.00	0.00	0.	*	2	1400	229	0.00	0.00	0.00	0.
1	1310	80	0.00	0.00	0.00	0.	*	2	1410	230	0.00	0.00	0.00	0.
1	1320	81	0.00	0.00	0.00	0.	*	2	1420	231	0.00	0.00	0.00	0.
1	1330	82	0.00	0.00	0.00	0.	*	2	1430	232	0.00	0.00	0.00	0.
1	1340	83	0.00	0.00	0.00	0.	*	2	1440	233	0.00	0.00	0.00	0.
1	1350	84	0.00	0.00	0.00	0.	*	2	1450	234	0.00	0.00	0.00	0.
1	1400	85	0.00	0.00	0.00	0.	*	2	1500	235	0.00	0.00	0.00	0.
1	1410	86	0.00	0.00	0.00	0.	*	2	1510	236	0.00	0.00	0.00	0.
1	1420	87	0.00	0.00	0.00	0.	*	2	1520	237	0.00	0.00	0.00	0.
1	1430	88	0.00	0.00	0.00	0.	*	2	1530	238	0.00	0.00	0.00	0.
1	1440	89	0.00	0.00	0.00	0.	*	2	1540	239	0.00	0.00	0.00	0.
1	1450	90	0.00	0.00	0.00	0.	*	2	1550	240	0.00	0.00	0.00	0.
1	1500	91	0.00	0.00	0.00	0.	*	2	1600	241	0.00	0.00	0.00	0.
1	1510	92	0.00	0.00	0.00	0.	*	2	1610	242	0.00	0.00	0.00	0.
1	1520	93	0.00	0.00	0.00	0.	*	2	1620	243	0.00	0.00	0.00	0.
1	1530	94	0.00	0.00	0.00	0.	*	2	1630	244	0.00	0.00	0.00	0.
1	1540	95	0.00	0.00	0.00	0.	*	2	1640	245	0.00	0.00	0.00	0.
1	1550	96	0.00	0.00	0.00	0.	*	2	1650	246	0.00	0.00	0.00	0.
1	1600	97	0.00	0.00	0.00	0.	*	2	1700	247	0.00	0.00	0.00	0.
1	1610	98	0.00	0.00	0.00	0.	*	2	1710	248	0.00	0.00	0.00	0.
1	1620	99	0.00	0.00	0.00	0.	*	2	1720	249	0.00	0.00	0.00	0.
1	1630	100	0.00	0.00	0.00	0.	*	2	1730	250	0.00	0.00	0.00	0.
1	1640	101	0.00	0.00	0.00	0.	*	2	1740	251	0.00	0.00	0.00	0.
1	1650	102	0.00	0.00	0.00	0.	*	2	1750	252	0.00	0.00	0.00	0.
1	1700	103	0.00	0.00	0.00	0.	*	2	1800	253	0.00	0.00	0.00	0.
1	1710	104	0.00	0.00	0.00	0.	*	2	1810	254	0.00	0.00	0.00	0.
1	1720	105	0.00	0.00	0.00	0.	*	2	1820	255	0.00	0.00	0.00	0.

1	1730	106	0.00	0.00	0.00	0.	*	2	1830	256	0.00	0.00	0.00	0.
1	1740	107	0.00	0.00	0.00	0.	*	2	1840	257	0.00	0.00	0.00	0.
1	1750	108	0.00	0.00	0.00	0.	*	2	1850	258	0.00	0.00	0.00	0.
1	1800	109	0.00	0.00	0.00	0.	*	2	1900	259	0.00	0.00	0.00	0.
1	1810	110	0.00	0.00	0.00	0.	*	2	1910	260	0.00	0.00	0.00	0.
1	1820	111	0.00	0.00	0.00	0.	*	2	1920	261	0.00	0.00	0.00	0.
1	1830	112	0.00	0.00	0.00	0.	*	2	1930	262	0.00	0.00	0.00	0.
1	1840	113	0.00	0.00	0.00	0.	*	2	1940	263	0.00	0.00	0.00	0.
1	1850	114	0.00	0.00	0.00	0.	*	2	1950	264	0.00	0.00	0.00	0.
1	1900	115	0.00	0.00	0.00	0.	*	2	2000	265	0.00	0.00	0.00	0.
1	1910	116	0.00	0.00	0.00	0.	*	2	2010	266	0.00	0.00	0.00	0.
1	1920	117	0.00	0.00	0.00	0.	*	2	2020	267	0.00	0.00	0.00	0.
1	1930	118	0.00	0.00	0.00	0.	*	2	2030	268	0.00	0.00	0.00	0.
1	1940	119	0.00	0.00	0.00	0.	*	2	2040	269	0.00	0.00	0.00	0.
1	1950	120	0.00	0.00	0.00	0.	*	2	2050	270	0.00	0.00	0.00	0.
1	2000	121	0.00	0.00	0.00	0.	*	2	2100	271	0.00	0.00	0.00	0.
1	2010	122	0.00	0.00	0.00	0.	*	2	2110	272	0.00	0.00	0.00	0.
1	2020	123	0.00	0.00	0.00	0.	*	2	2120	273	0.00	0.00	0.00	0.
1	2030	124	0.00	0.00	0.00	0.	*	2	2130	274	0.00	0.00	0.00	0.
1	2040	125	0.00	0.00	0.00	0.	*	2	2140	275	0.00	0.00	0.00	0.
1	2050	126	0.00	0.00	0.00	0.	*	2	2150	276	0.00	0.00	0.00	0.
1	2100	127	0.00	0.00	0.00	0.	*	2	2200	277	0.00	0.00	0.00	0.
1	2110	128	0.00	0.00	0.00	0.	*	2	2210	278	0.00	0.00	0.00	0.
1	2120	129	0.00	0.00	0.00	0.	*	2	2220	279	0.00	0.00	0.00	0.
1	2130	130	0.00	0.00	0.00	0.	*	2	2230	280	0.00	0.00	0.00	0.
1	2140	131	0.00	0.00	0.00	0.	*	2	2240	281	0.00	0.00	0.00	0.
1	2150	132	0.00	0.00	0.00	0.	*	2	2250	282	0.00	0.00	0.00	0.
1	2200	133	0.00	0.00	0.00	0.	*	2	2300	283	0.00	0.00	0.00	0.
1	2210	134	0.00	0.00	0.00	0.	*	2	2310	284	0.00	0.00	0.00	0.
1	2220	135	0.00	0.00	0.00	0.	*	2	2320	285	0.00	0.00	0.00	0.
1	2230	136	0.00	0.00	0.00	0.	*	2	2330	286	0.00	0.00	0.00	0.
1	2240	137	0.00	0.00	0.00	0.	*	2	2340	287	0.00	0.00	0.00	0.
1	2250	138	0.00	0.00	0.00	0.	*	2	2350	288	0.00	0.00	0.00	0.
1	2300	139	0.00	0.00	0.00	0.	*	3	0000	289	0.00	0.00	0.00	0.
1	2310	140	0.00	0.00	0.00	0.	*	3	0010	290	0.00	0.00	0.00	0.
1	2320	141	0.00	0.00	0.00	0.	*	3	0020	291	0.00	0.00	0.00	0.
1	2330	142	0.00	0.00	0.00	0.	*	3	0030	292	0.00	0.00	0.00	0.
1	2340	143	0.00	0.00	0.00	0.	*	3	0040	293	0.00	0.00	0.00	0.
1	2350	144	0.00	0.00	0.00	0.	*	3	0050	294	0.00	0.00	0.00	0.
2	0000	145	0.00	0.00	0.00	0.	*	3	0100	295	0.00	0.00	0.00	0.
2	0010	146	0.00	0.00	0.00	0.	*	3	0110	296	0.00	0.00	0.00	0.
2	0020	147	0.00	0.00	0.00	0.	*	3	0120	297	0.00	0.00	0.00	0.
2	0030	148	0.00	0.00	0.00	0.	*	3	0130	298	0.00	0.00	0.00	0.
2	0040	149	0.00	0.00	0.00	0.	*	3	0140	299	0.00	0.00	0.00	0.
2	0050	150	0.00	0.00	0.00	0.	*	3	0150	300	0.00	0.00	0.00	0.
							*							

****

TOTAL RAINFALL = 3.28, TOTAL LOSS = 1.32, TOTAL EXCESS = 1.96

CALCULATION OF Tc & R

Calculated by: ____________________ Date: ____________

Checked by: ____________________ Project: ____________

Watershed: ______________________________

Rainfall Frequency: 100 - yr Duration: 6 - hr. Pattern #: ________

Rainfall Loss Method:
[X] Green & Ampt Method
[ ] IL + ULR by soil texture
[ ] IL + ULR by hydrologic soil group

Tabulate Period of Peak Rainfall Excess

Clock Time @ end of Increm.	Increm. Excess in.
320	0.06
330	0.07
340	0.30
350	0.36
360	0.41
370	0.21
380	0.14
390	0.06

Rearrange Incremental Excesses in Order of Decreasing Average Intensity

Accum. Time hr./min	Increm. Excess in.	Accum. Excess in.	Avg. Excess Intensity in./hr.
10	0.41	0.41	2.40
20	0.36	0.77	2.31
30	0.30	1.07	2.14
40	0.21	1.28	1.92
50	0.14	1.42	1.70
60	0.07	1.49	1.49
70	0.06	1.55	1.33
80	0.06	1.61	1.21

A = __________ sq.mi.

L = __________ mi.

S = __________ ft/mi.

$Kb = m\,[\log(A * 640)] + b$

$Kb = (\quad)\,\log(\quad *640) + (\quad)$

$Kb =$ __________

$Tc = 11.4\,L^{.50}\,Kb^{.52}\,S^{-.31}\,i^{-.38}$

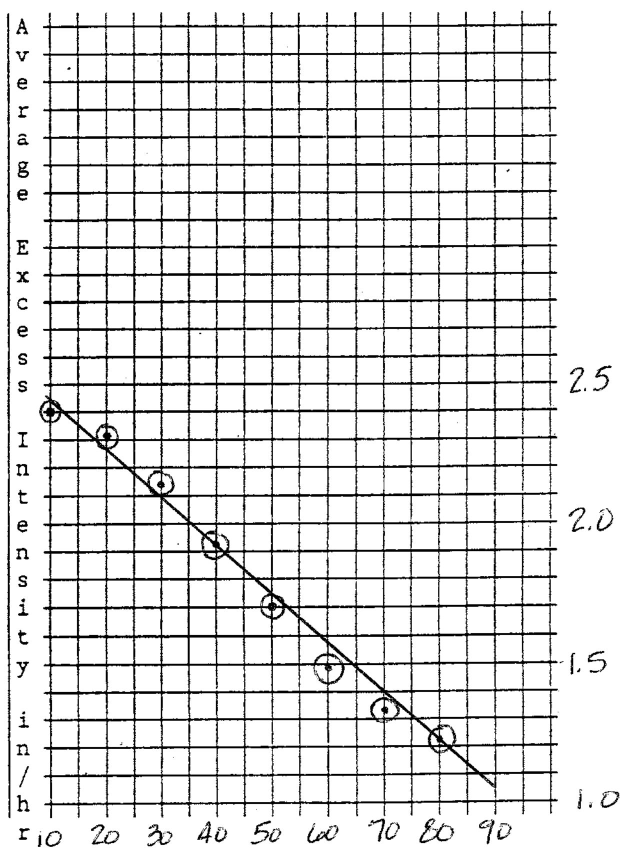
$Tc = (\_\_\_\_\_\_)\,i^{-.38}$

Trial Tc hrs	i	Calc. Tc hrs
0.667	1.92	0.754
0.754	1.82	0.770
0.770	1.81	0.771

$\boxed{Tc = 0.771 \text{ hr.}}$

$R = .37\,Tc^{1.11}\,A^{-.57}\,L^{.80}$

$\boxed{R = 0.368 \text{ hr.}}$



Using the above procedure, T_c and R for each of the Clark subbasins are as follows:

Subbasin ID	T_c hours	R hours
S2	0.771	0.368
S3	0.404	0.204
S4	0.363	0.196
S5	0.368	0.239
S6	0.367	0.211
S7	0.417	0.198
S8	0.438	0.301
S9	0.300	0.135
S10	0.208	0.158
S11	0.854	0.529

f. Select the time-area relation for each subbasin.

The majority of the land in subbasins S2 and S11 is undeveloped, therefore use the natural time-area relation. Use the urban time-area relation for all other Clark subbasins.

9.5 CHANNEL ROUTING

9.5.1 Application of Normal-Depth Routing

1. Routing reaches should have relatively constant characteristics along the entire reach (i.e. geometry, slope, roughness, etc). If not, then consider subdividing the reach.

2. Too short of a routing reach may cause numeric instabilities and/or increase the peak discharge. The model output should be checked for unstable warning messages. If unstable warning messages are reported, then check the discharge range of instability in comparison to the peak discharge and plot the hydrograph for inspection.

3. If several short routing reaches occur in succession and attenuation is anticipated, then the channel routing operation can be replaced by a hydrograph lag operation.

4. Channel geometry must have sufficient capacity to convey the peak discharge.

5. The number of computational subreaches (NSTPS), should correspond to the lag time computed by HEC-1 for the routing reach. Example:

   An inflow hydrograph with a time to peak of 4.5 hrs is routed down a 5000 ft natural channel. The estimated NSTPS is 2 and NMIN is set to 5 min. The resulting time to peak of the routing operation is 4.92 hours, a lag of 25 minutes. The actual NSTPS should be (lag/NMIN)=5. This is an interactive process that should be repeated until NSTPS*NMIN approximates the lag.

9.5.2 Application of Kinematic Wave Routing

1. Kinematic Wave routing is most appropriately used where peak attenuation and channel transmission losses are not expected to be significant. The usual applications are for defined urban channels and short, steep natural channels, with minimal overbank flow.

2. When working with Kinematic Wave routing, channel capacity must be checked to assure proper conveyance of flow prior to the HEC-1 run. Otherwise, if the channel is undersized, the program will automatically extend channel boundaries to contain the flow.

3. The guidance, comments, and warnings in the HEC-1 User’s Manual should be studied and carefully observed in applying the Kinematic Wave method.

9.5.3 Application of Muskingum Routing

1. The Muskingum Routing method can be used where flood peak attenuation is expected. The best application of this method is for larger rivers with relatively flat slopes.

2. The parameters, K and X, are best determined by the analysis of stream gage data, if available. Where such data are available, K and X can be determined by analytic methods as presented in many hydrology textbooks, or the HEC-1 parameter optimization option can be used. Other regional flood studies (by the U.S. Army Corps of Engineers and others) may contain the results of such analyses for larger rivers in the County.

3. The following parameter estimation procedures apply primarily to natural stream channels which convey a significant amount of flow in the overbank areas during design-frequency events:

   a. NSTPS: The choice of a number of subreaches for a particular stream reach can be checked for computational stability using the following equation from the HEC-1 Manual:

$$\frac{1}{(1-X)} \leq \frac{K \times 60}{NSTPS \times NMIN} \leq \frac{1}{2(X)}$$

   where:

K	=	the travel time through the entire reach, in hours,
X	=	Muskingum 'X',
NMIN	=	the computational time step, (in hours) and
NSTPS	=	the integer number of subreaches.

   b. K: K is the travel time of the floodwave peak through the entire reach. Calculation using Manning's equation is usually an appropriate method for estimating the floodwave velocity, V_m, with the following provisions:

      i. Use an average channel area and wetted perimeter for the reach, assuming bankfull conditions.

      ii. Choose an 'n' value representative of the main channel only. Do not include the overbank roughness in a weighted average.

iii. Calculate an average flow velocity for the reach (V).

iv. Use the following ratios (Cudworth, 1989) to estimate V_m, the velocity of the floodwave:

Channel Geometry	$\frac{V_m}{V}$
Wide rectangular	1.67
Wide parabolic	1.44
Triangular	1.33

The value of K is then estimated by dividing the reach length by V_m.

c. X: For wide, shallow channels with low to moderate slopes and significant overbank flow during the design flood being modeled, choose X = 0.15 to 0.25. For steep to very steep, narrow, deep channels with little overbank flow, choose X = 0.25 to 0.40.

9.5.4 Application of Muskingum-Cunge Routing

1. For constructed channels and some natural channels, this routing option can be used by providing all input on the RD record only. This requires selection of a predetermined channel shape (see the HEC-1 User's Manual). Complex channel geometry and/or variable channel roughness (channel and overbank) can be modeled with the additional use of RC, RX and RY records. An eight-point cross section is input on the RX and RY records to describe the representative channel geometry.

2. Execution of the HEC-1 program may terminate with a math error message if the inflow to the routing reach is zero (no runoff generated from the upstream watershed). This may occur in situations that have either very low rainfall depth (intensities) or exceptionally high rainfall losses, or zero diversion (most often).

9.6 INDIRECT METHODS

9.6.1 Procedures

The following instructions should be followed for verifying peak discharges that are derived by analytic methods, (Rational Method or rainfall-runoff modeling).

A. Verification with Unit Peak Discharge Curves:

1. For a given watershed of drainage area (A), in square miles, divide the 100-year primary peak discharge estimate by A.

2. Plot the unit peak discharge on a copy of Figure 8.1. Note the location of the plotted point in relation to the various curves in that figure.

B. Verification with USGS Data for Arizona:

1. Calculate the 100-year peak discharge estimate by Equation 8.1

2. Select Figure 8.3 or 8.4 according to watershed drainage area size, and plot the 100-year peak discharge estimate on a copy of that figure.

3. Using watershed drainage area as a guide, identify gaged watersheds of the same approximate size from Table 8.1. Tabulate the peak discharge statistics and watershed characteristics for those gaged watersheds by using the USGS report (Garrett and Gellenbeck, 1991). Compare these to the computed peak discharge estimates and watershed characteristics for the watershed of interest.

C. Verification with Regional Regression Equations:

1. Calculate the mean basin elevation ($ELEV$). This can be done by placing a transparent grid over the largest scale topographic map available. The grid spacing should be selected such that at least 20 elevation points are sampled. The elevation at each grid point is determined and the elevations are then averaged.

2. Determine the flood region (Figure 8.6).

3. Check the drainage area using the appropriate scatter diagram to determine if the values are in the “cloud of common values.” Proceed with the analysis regardless of the outcome, but clearly note if the variable values are not within the “cloud of common values.”

4. Calculate the peak discharge estimates using the applicable regression equations for the flood region within which the project site is located.

5. Plot the 100-year peak discharge estimate on a copy of the appropriate Q_{100} data points and 100-year peak discharge relation graph (Figure 8.8 or 8.9).

D. For all three Indirect Methods:

1. Quantitatively and qualitatively analyze the results of the primary and the secondary peak discharge estimates. Address watershed characteristics that may explain differences between the primary and secondary estimates.

2. Prepare a summary of results by all methods and a qualitative evaluation of the results.

10 REFERENCES

10.1 REFERENCES

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APPENDICES

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APPENDIX A: RAINFALL

A.1 Section 1: Isopluvial Maps

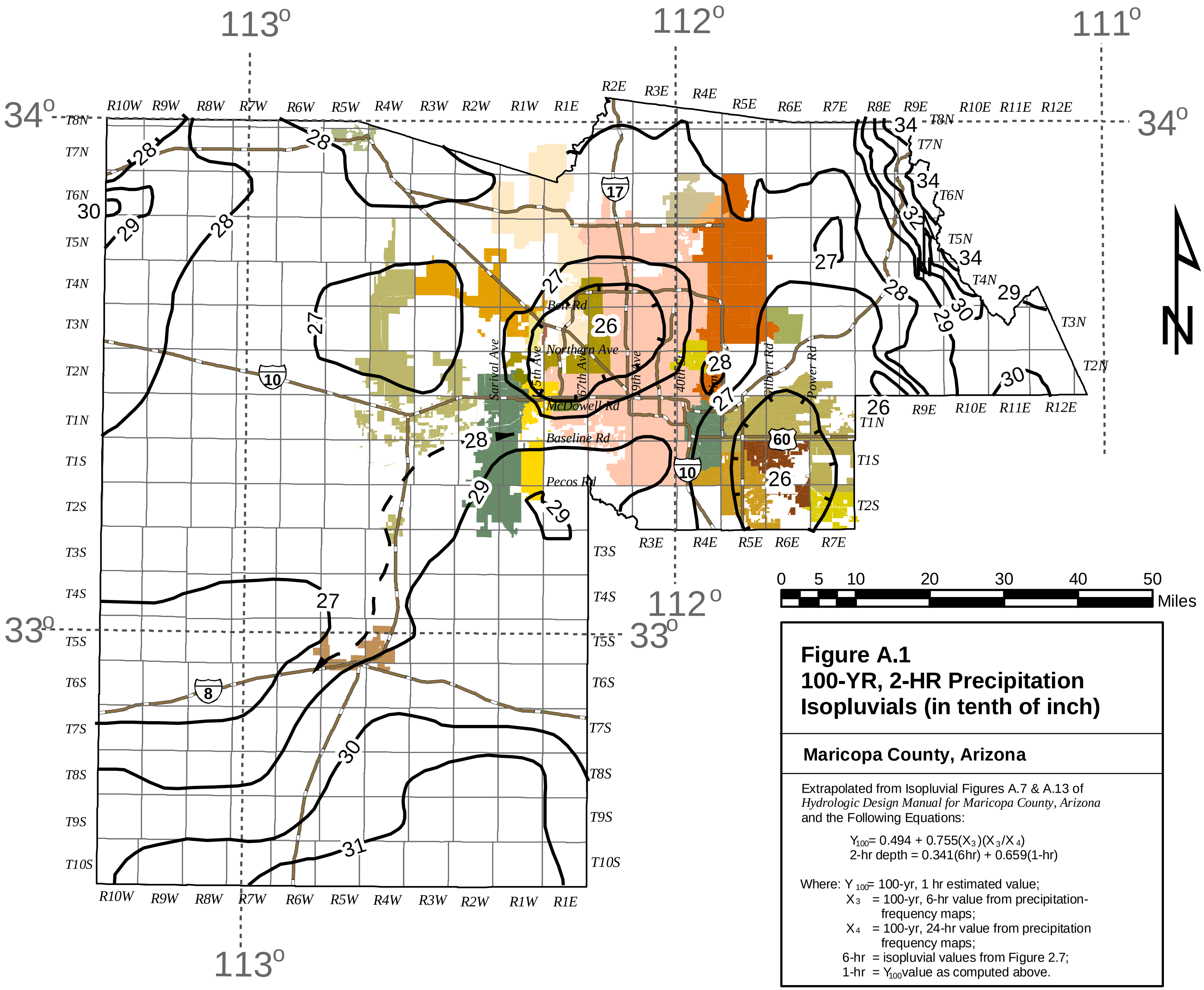

Figure A.1
100-YR, 2-HR Precipitation Isopluvials (in tenth of inch)

Maricopa County, Arizona

Extrapolated from Isopluvial Figures A.7 & A.13 of *Hydrologic Design Manual for Maricopa County, Arizona* and the Following Equations:

$Y_{100} = 0.494 + 0.755(X_3)(X_3/X_4)$
2-hr depth = 0.341(6hr) + 0.659(1-hr)

Where: Y_{100} = 100-yr, 1 hr estimated value;
X_3 = 100-yr, 6-hr value from precipitation-frequency maps;
X_4 = 100-yr, 24-hr value from precipitation frequency maps;
6-hr = isopluvial values from Figure 2.7;
1-hr = Y_{100} value as computed above.

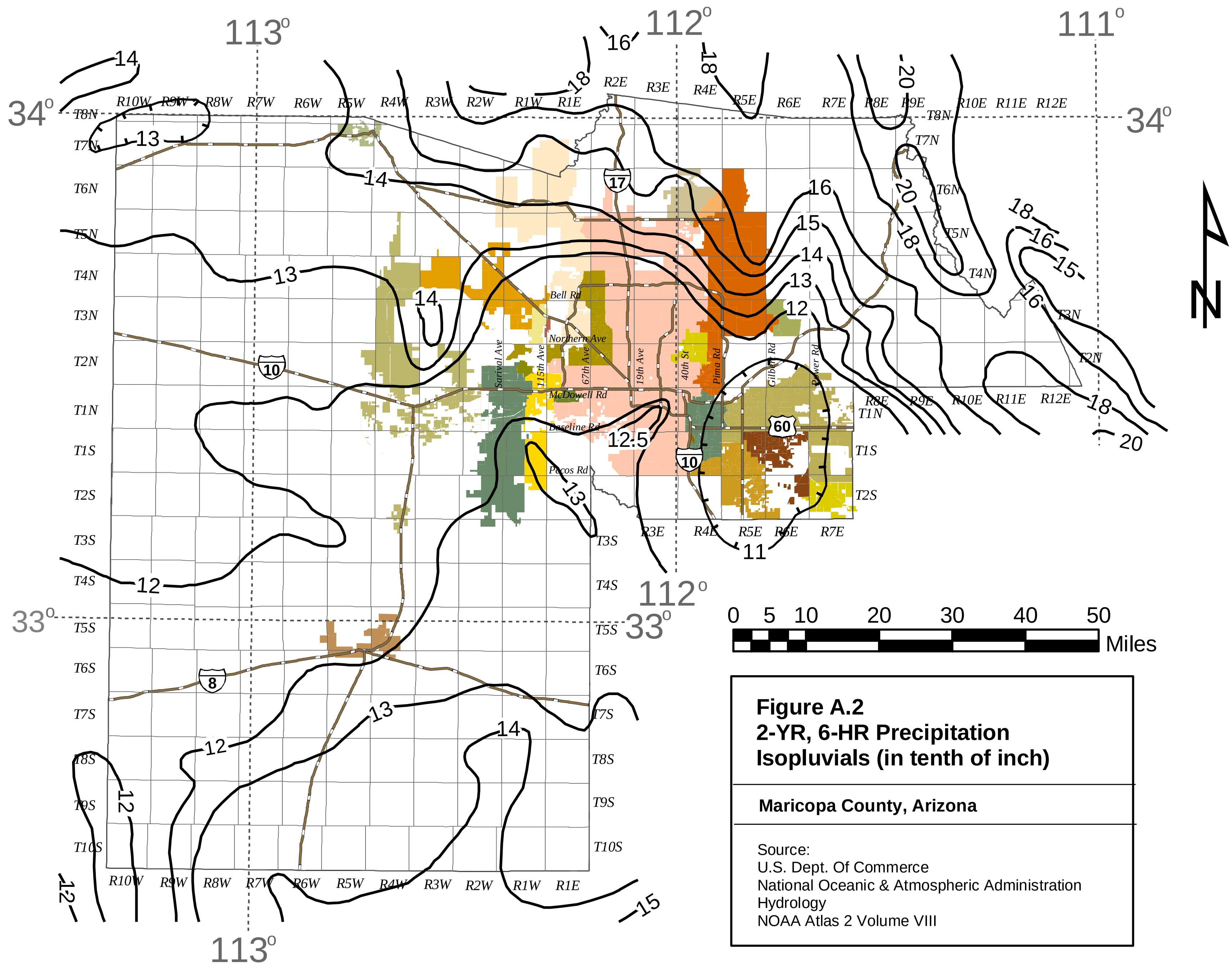

Figure A.2
2-YR, 6-HR Precipitation Isopluvials (in tenth of inch)

Maricopa County, Arizona

Source:
U.S. Dept. Of Commerce
National Oceanic & Atmospheric Administration
Hydrology
NOAA Atlas 2 Volume VIII

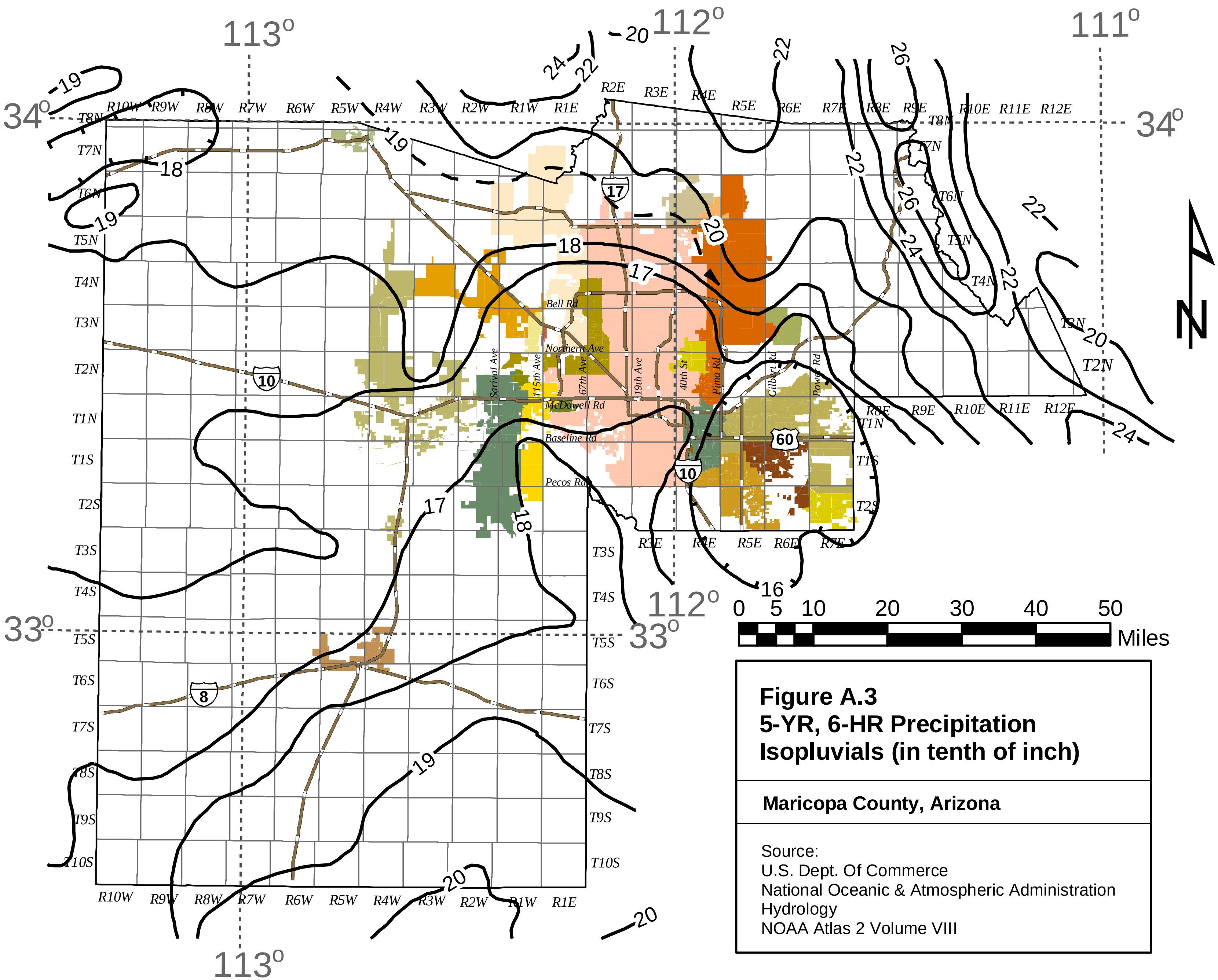

Figure A.3
5-YR, 6-HR Precipitation Isopluvials (in tenth of inch)

Maricopa County, Arizona

Source:
U.S. Dept. Of Commerce
National Oceanic & Atmospheric Administration
Hydrology
NOAA Atlas 2 Volume VIII

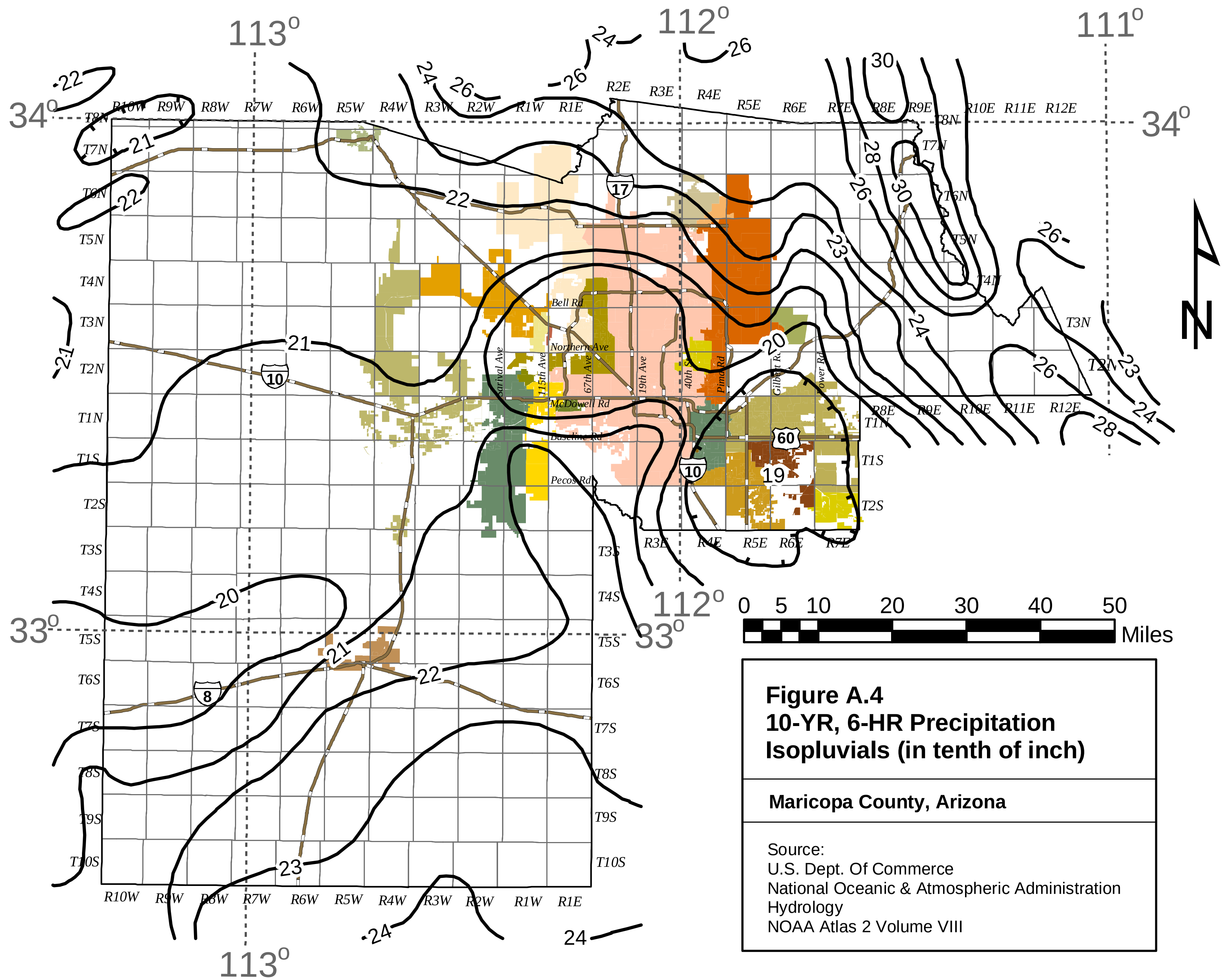

Figure A.4
10-YR, 6-HR Precipitation Isopluvials (in tenth of inch)

Maricopa County, Arizona

Source:
U.S. Dept. Of Commerce
National Oceanic & Atmospheric Administration
Hydrology
NOAA Atlas 2 Volume VIII

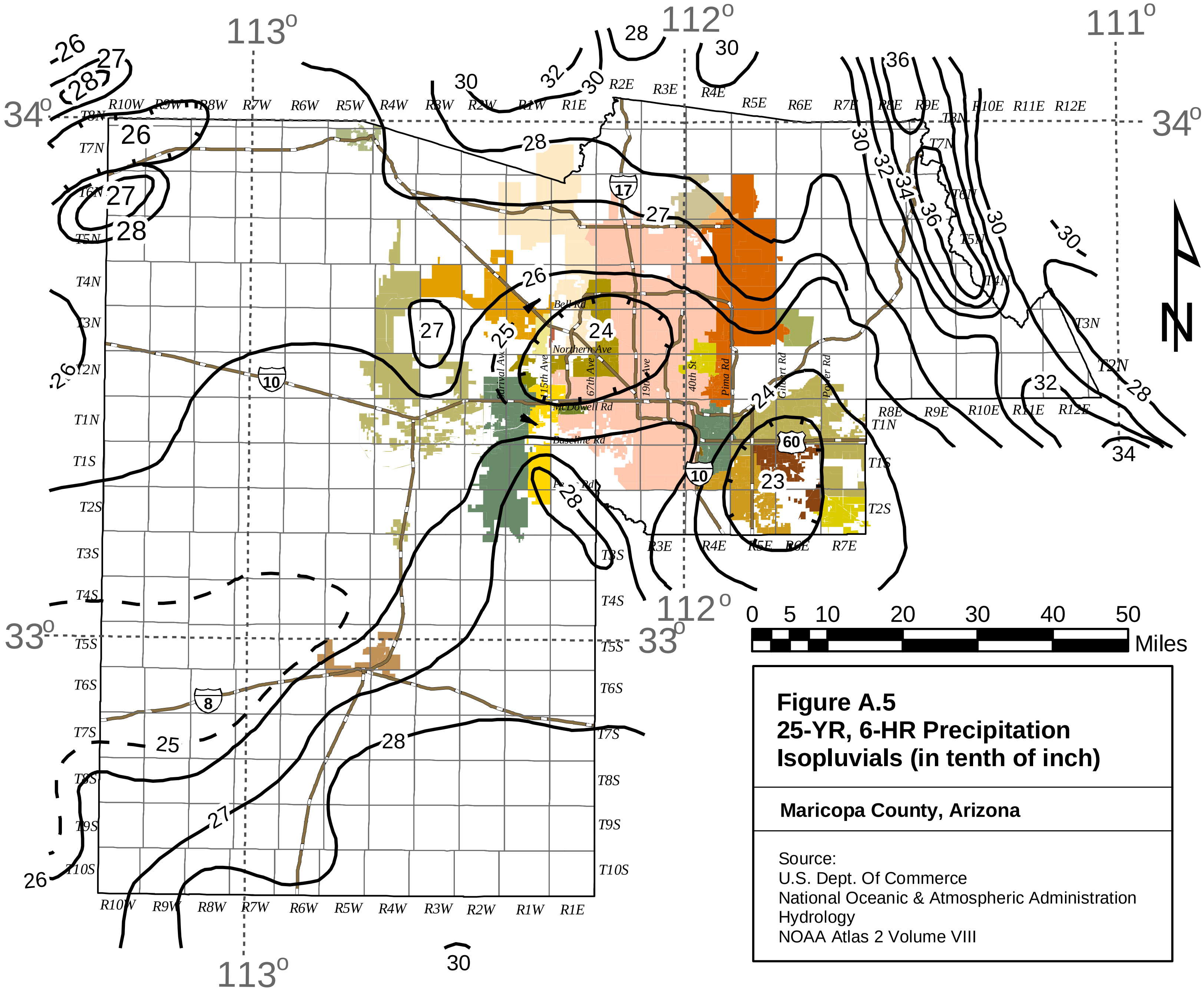

Figure A.5
25-YR, 6-HR Precipitation Isopluvials (in tenth of inch)

Maricopa County, Arizona

Source:
U.S. Dept. Of Commerce
National Oceanic & Atmospheric Administration
Hydrology
NOAA Atlas 2 Volume VIII

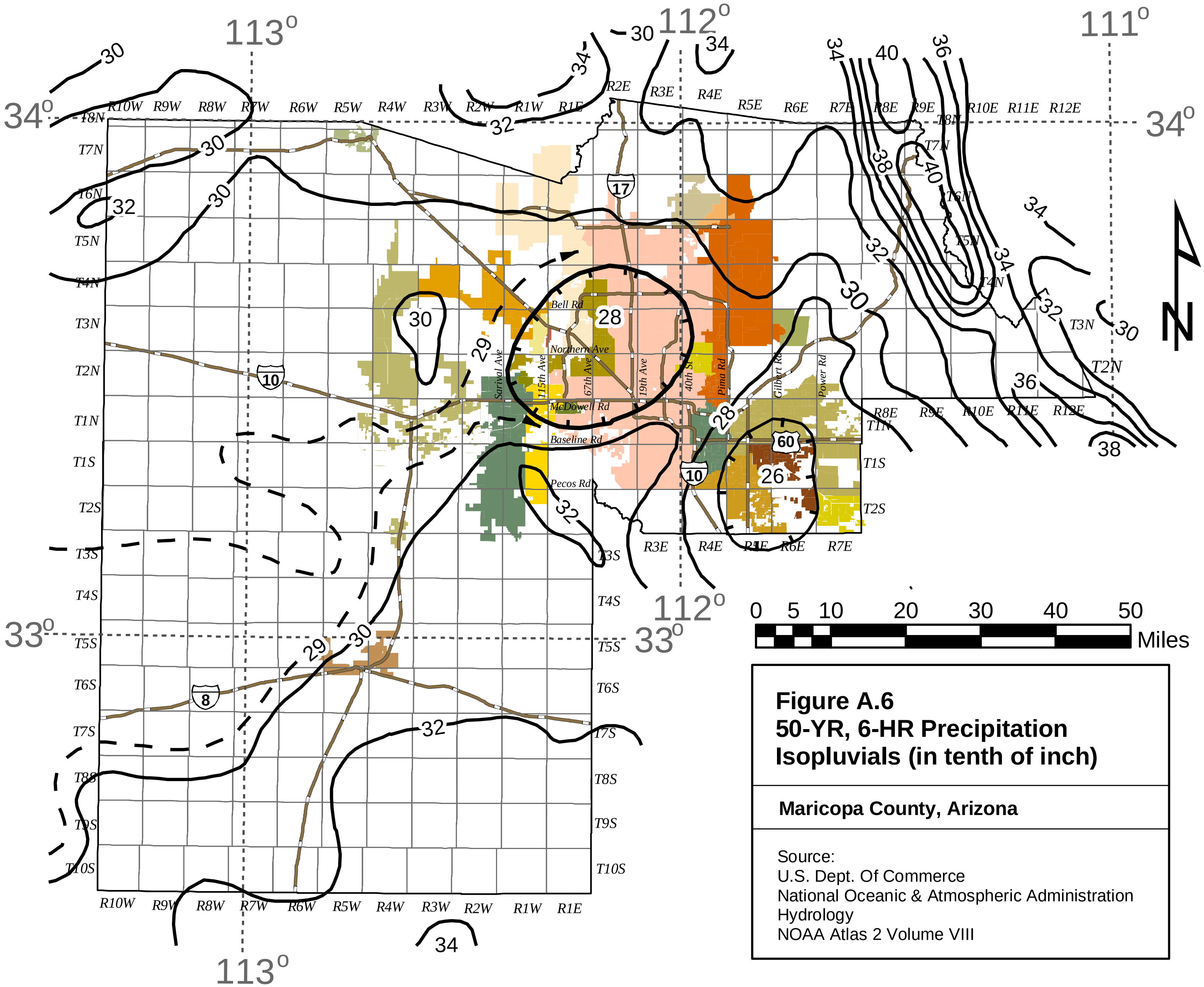

Figure A.6
50-YR, 6-HR Precipitation Isopluvials (in tenth of inch)

Maricopa County, Arizona

Source:
U.S. Dept. Of Commerce
National Oceanic & Atmospheric Administration
Hydrology
NOAA Atlas 2 Volume VIII

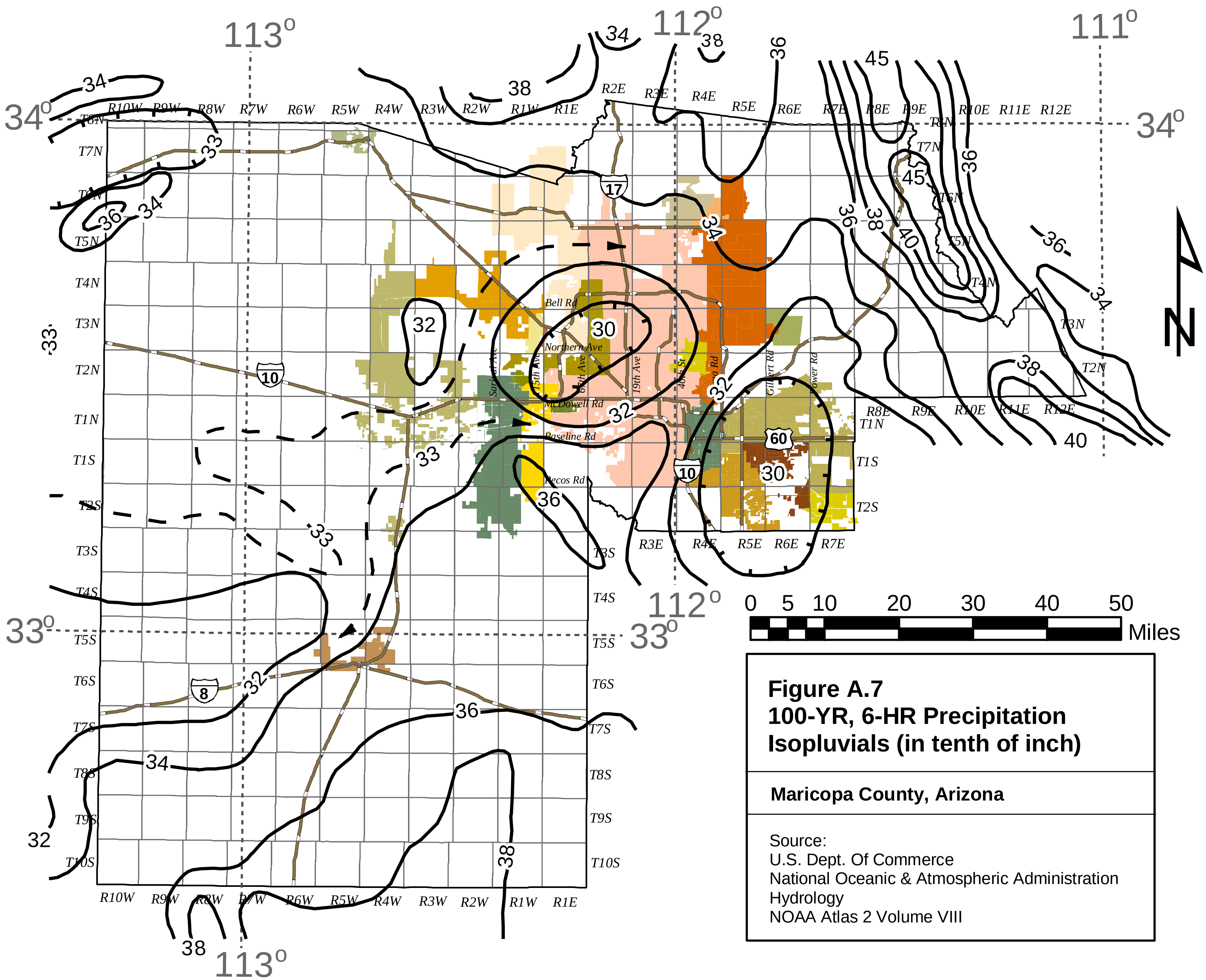

Figure A.7
100-YR, 6-HR Precipitation Isopluvials (in tenth of inch)

Maricopa County, Arizona

Source:
U.S. Dept. Of Commerce
National Oceanic & Atmospheric Administration
Hydrology
NOAA Atlas 2 Volume VIII

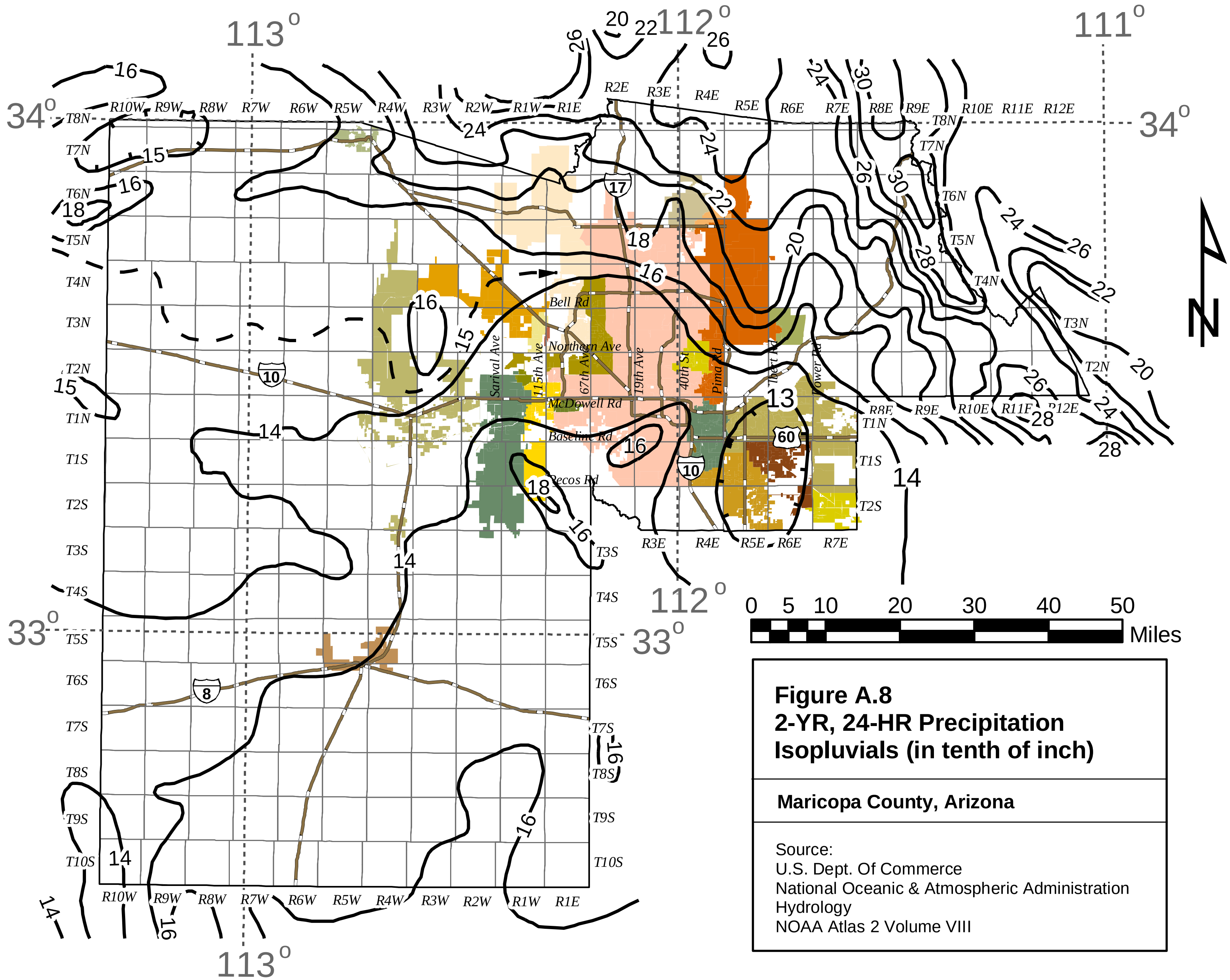

Figure A.8
2-YR, 24-HR Precipitation Isopluvials (in tenth of inch)

Maricopa County, Arizona

Source:
U.S. Dept. Of Commerce
National Oceanic & Atmospheric Administration
Hydrology
NOAA Atlas 2 Volume VIII

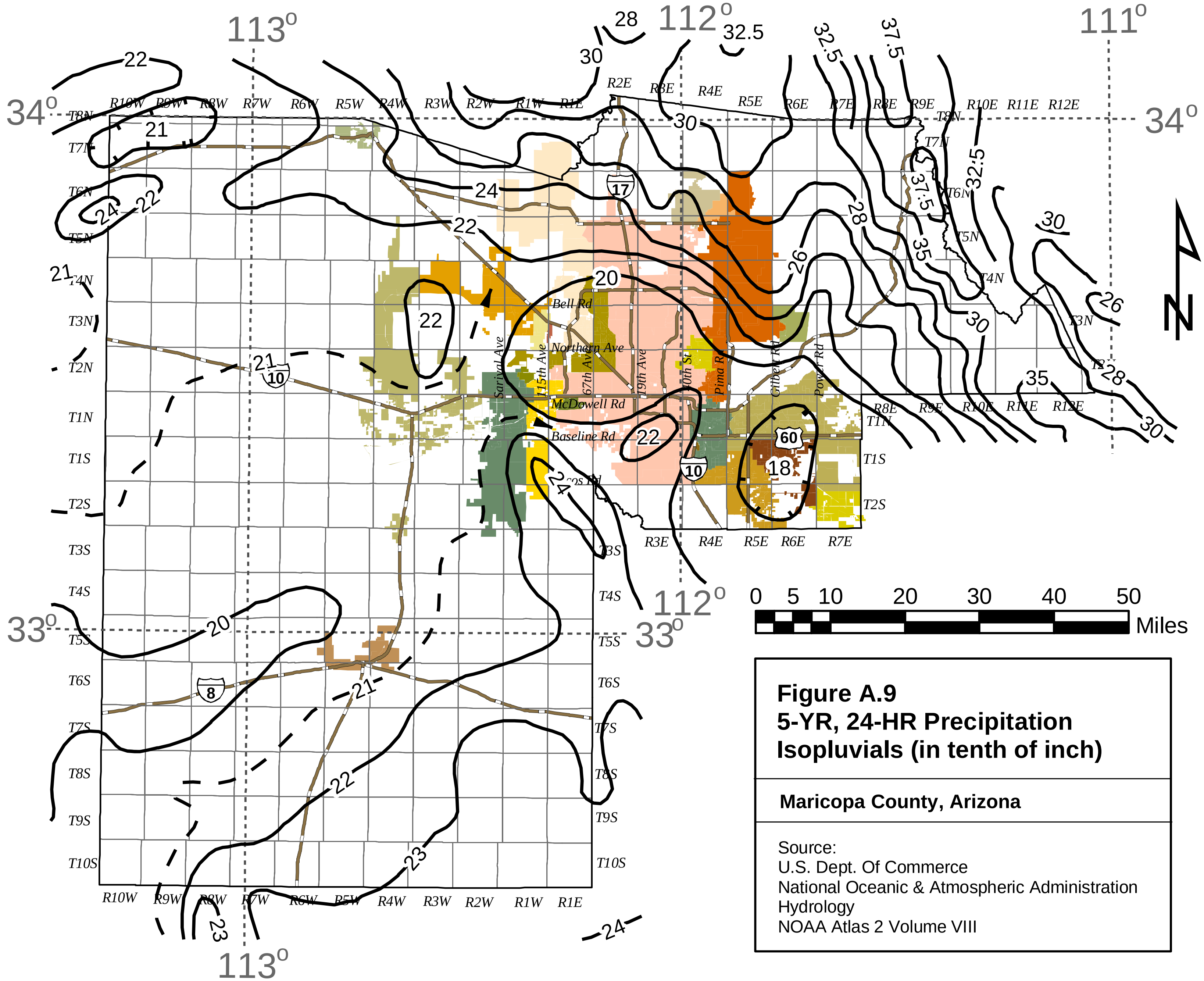

Figure A.9
5-YR, 24-HR Precipitation Isopluvials (in tenth of inch)

Maricopa County, Arizona

Source:
U.S. Dept. Of Commerce
National Oceanic & Atmospheric Administration
Hydrology
NOAA Atlas 2 Volume VIII

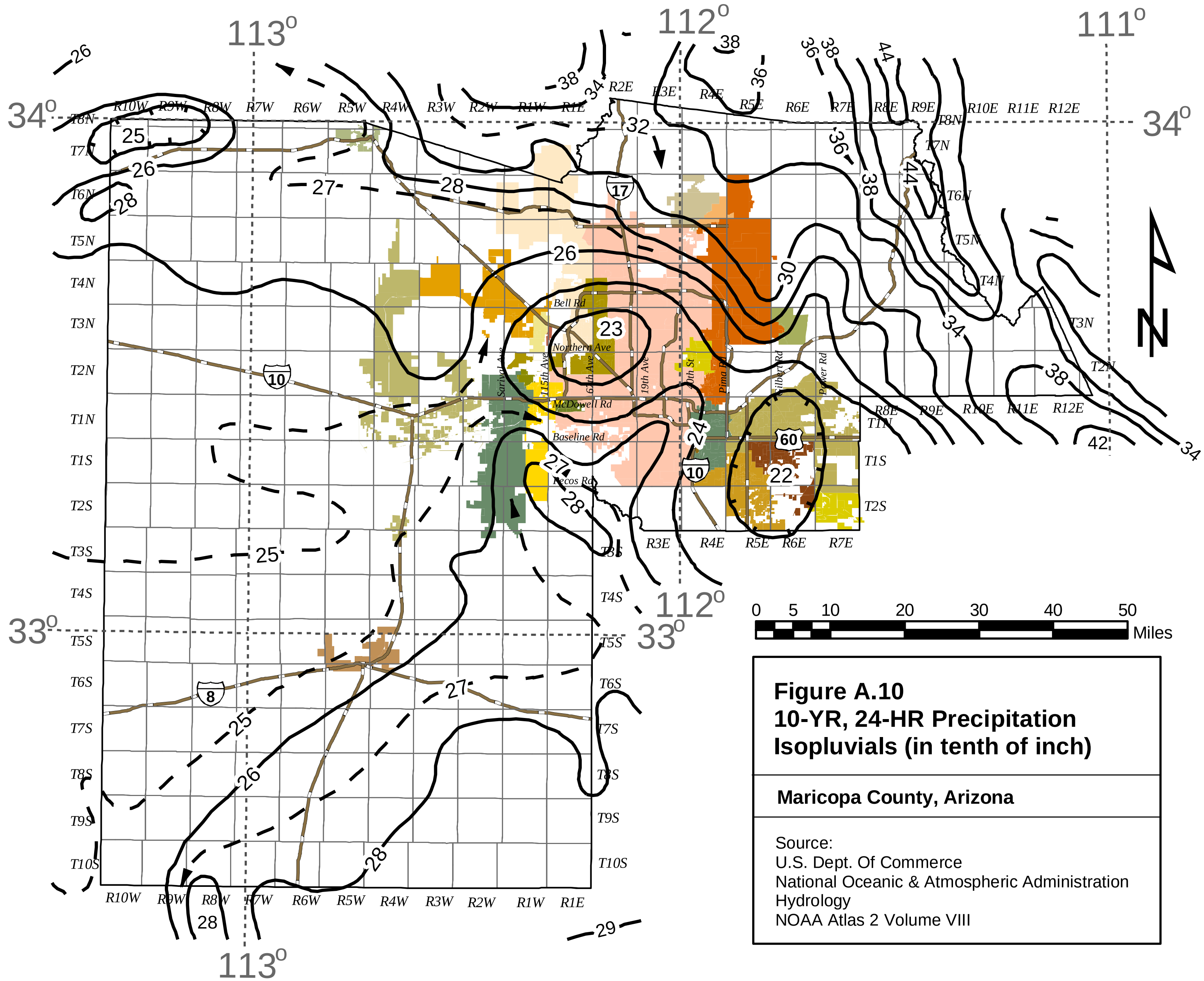

Figure A.10
10-YR, 24-HR Precipitation Isopluvials (in tenth of inch)

Maricopa County, Arizona

Source:
U.S. Dept. Of Commerce
National Oceanic & Atmospheric Administration
Hydrology
NOAA Atlas 2 Volume VIII

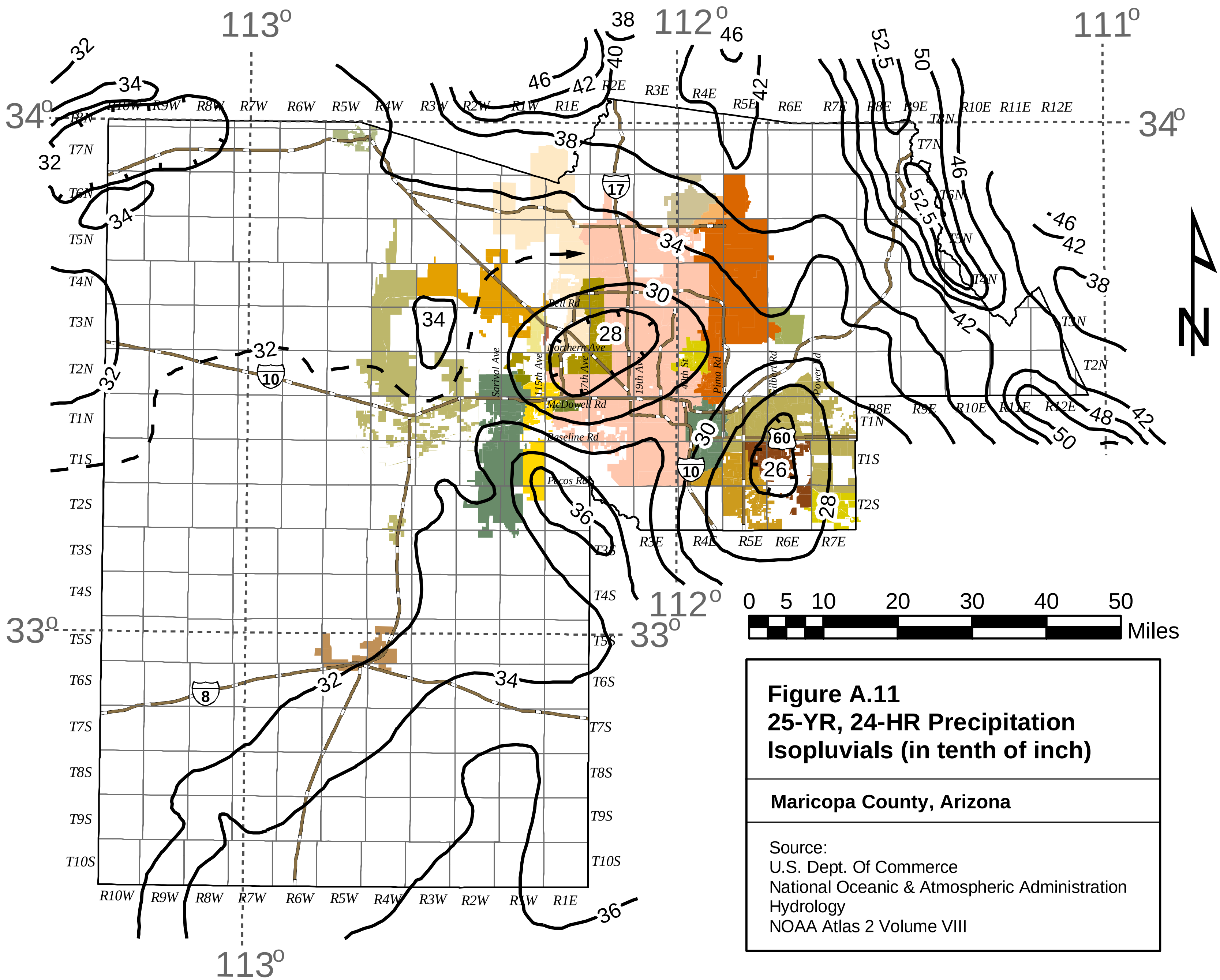

Figure A.11
25-YR, 24-HR Precipitation Isopluvials (in tenth of inch)

Maricopa County, Arizona

Source:
U.S. Dept. Of Commerce
National Oceanic & Atmospheric Administration
Hydrology
NOAA Atlas 2 Volume VIII

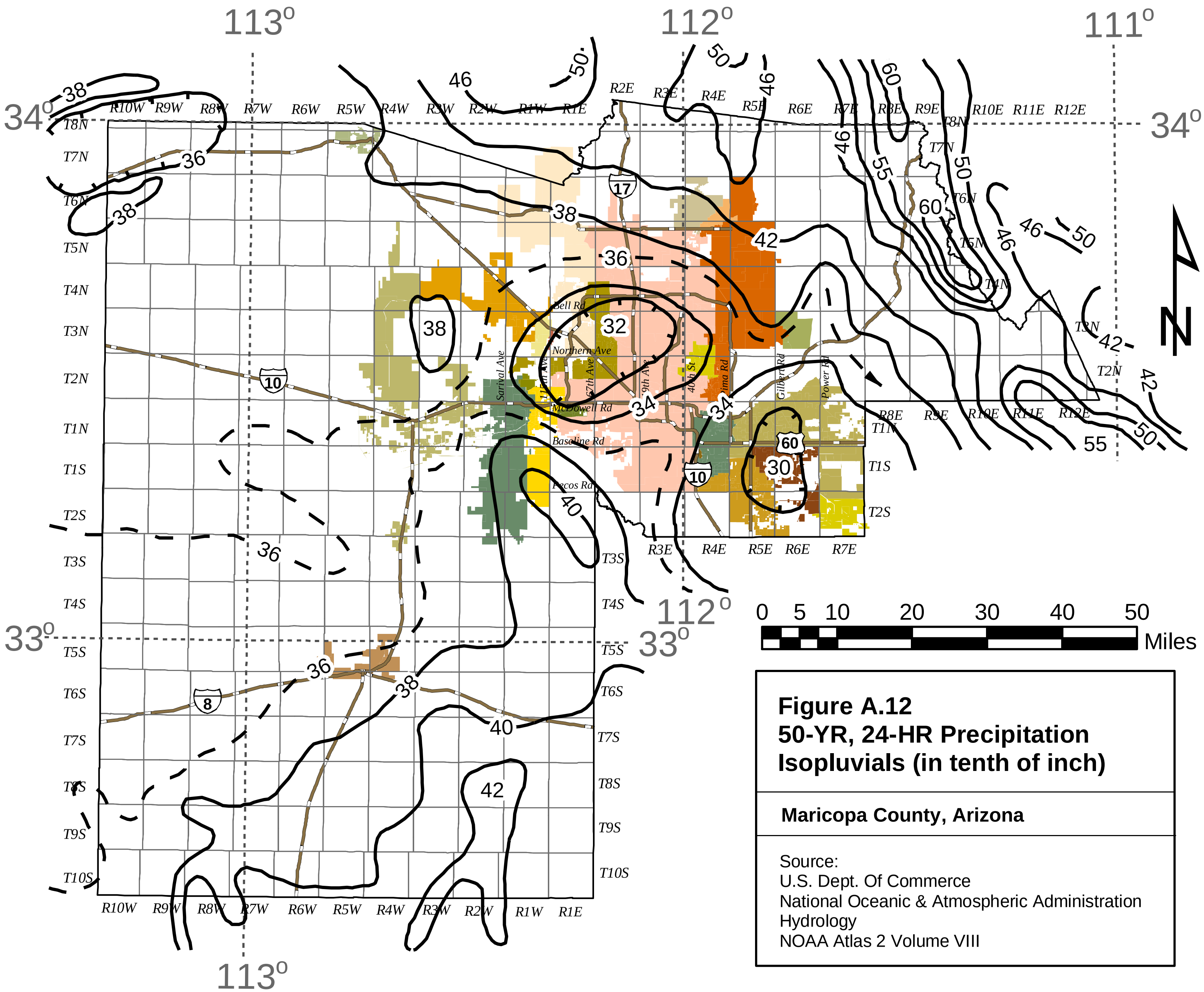

Figure A.12
50-YR, 24-HR Precipitation Isopluvials (in tenth of inch)

Maricopa County, Arizona

Source:
U.S. Dept. Of Commerce
National Oceanic & Atmospheric Administration
Hydrology
NOAA Atlas 2 Volume VIII

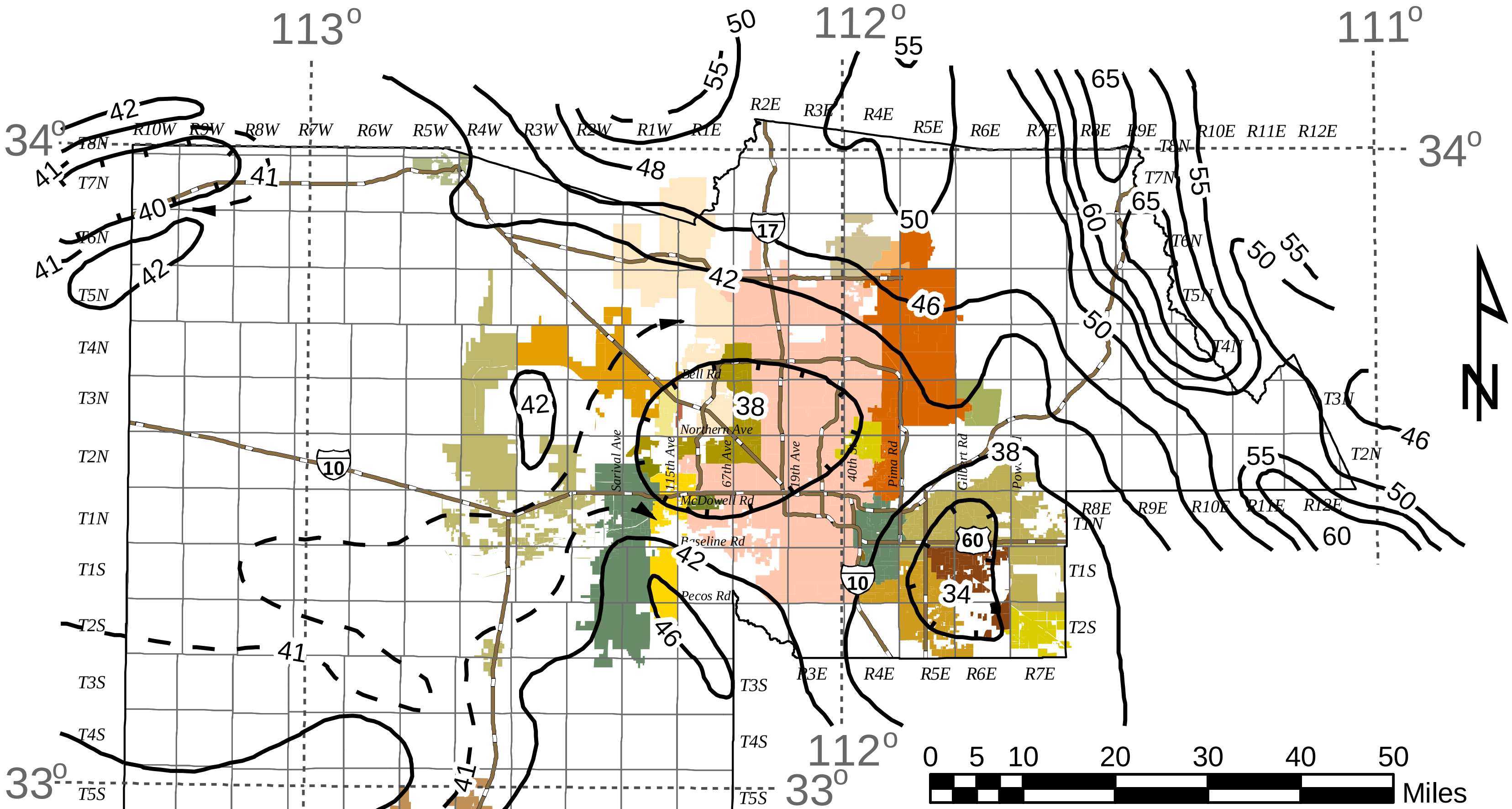

Figure A.13
100-YR, 24-HR Precipitation Isopluvials (in tenth of inch)

Maricopa County, Arizona

Source:
U.S. Dept. Of Commerce
National Oceanic & Atmospheric Administration
Hydrology
NOAA Atlas 2 Volume VIII

APPENDIX A: RAINFALL

A.2 Section 2: Precipitation Depth-Duration Figure

APPENDIX A: RAINFALL

A.3 Section 3: PREFRE Manual

* P R E F R E *

COMPUTATION OF PRECIPITATION FREQUENCY-DURATION VALUES

IN THE WESTERN UNITED STATES

PROGRAM USER MANUAL

FLOOD SECTION
SURFACE WATER BRANCH
EARTH SCIENCES DIVISION
BUREAU OF RECLAMATION

DENVER, COLORADO

AUGUST 1988

USER MANUAL FOR PROGRAM PREFRE

--

COMPUTATION OF PRECIPITATION FREQUENCY-DURATION VALUES IN THE WESTERN UNITED STATES

1. Introduction.

The PREFRE computer program was written to compute the precipitation frequency values for each of 10 durations and for each of 7 return periods. This document describes how to prepare the input data, how to execute the program, and gives an example of the output.

The PREFRE program computes frequency values for 5-, 10-, 15-, and 30-minute and 1-, 2-, 3-, 6-, 12-, and 24-hour durations for return periods of 2, 5, 10, 25, 50, 100, and 500 years for areas in the 11 western states and presents the results in tabular form. It uses as input the precipitation frequency values taken from the NOAA Atlas 2 (11 volumes). The PREFRE program also duplicates the values in Weather Bureau Technical Paper No. 40 for the six Plains states within the Bureau's area of operations not included in the NOAA Atlas 2 volumes.

NOAA Atlas 2 reflects the effects of topography on precipitation frequencies, but it contains isohyetal maps for return periods of 2, 5, 10, 25, 50, and 100 years but only for 6- and 24-hour durations. For other durations, it is necessary to use the nomograms and equations included in the atlas.

The computer program was originally developed by Mr. Ralph Frederick, Office of Hydrology, NWS (National Weather Service). The program was extensively revised to fit Bureau of Reclamation needs in 1975 by Mr. James Mumford of what was then the Flood and Sedimentation Section, Engineering and Research Center. It was further revised in 1988 by Mr. Richard Eddy of the Flood Section to incorporate updated information for short-duration values.

The program is written in FORTRAN V for the Bureau's CYBER mainframe computer. This version has also been converted to FORTRAN 77 for use with personal computers (IBM compatible).

2. Input Data.

The following data are required for the program input file:

a. Site name.

b. Primary zone number identifying where the site is located, obtained from the map included as appendix A in this manual. The zone boundaries correspond to those found in NOAA Atlas 2, but the numbers may be different. It is advisable to identify the location of a site from the zone map in the atlas volume and refer to appendix A for the zone number used in PREFRE.

c. Zone number for short-duration values (appendix B).

d. Site latitude and longitude (required for primary zones 3, 9, and 11; optional for other primary zones).

e. Site elevation (required for primary zones 1, 2, and 6; optional for other primary zones).

f. NOAA Atlas 2 precipitation values (note that Atlas values are in tenths of inches).

(1) Standard: Enter the values of 2-year and 100-year return periods for durations of 6 hours and 24 hours.

(2) Option: The original NWS program was designed to input 12 precipitation frequency values. This format has been retained as an option. The 2-, 5-, 10-, 25-, 50-, and 100-year values for durations of 6 hours and 24 hours must be used as input for this option. The program uses the six return-period values and develops a line of best fit to the points read from the NOAA Atlas 2 maps. It then uses this line of best fit to recompute the return-period values and uses these computed values in all subsequent computations.

The input data format is presented in appendixes C1 through C3. Each field in a line must be separated from the next field by either a blank or a comma, and an entry is required for each field (i.e., enter zeroes if latitude, longitude, and elevation are omitted). Input data can be all metric, if desired.

3. Output Data.

The site name, zone numbers, and latitude, longitude, and elevation (if included in the input data) are printed as a heading. A table is then given showing the precipitation values for 2-, 5-, 10-, 25-, 50-, 100-, and 500-year return periods for durations of 5, 10, 15, and 30 minutes and 1, 2, 3, 6, 12, and 24 hours. Output units are the same as the input units. The PC version also prints the input data for reference. Appendix D1 is a sample output from the CYBER version of PREFRE. Appendix D2 is the standard PC output. Appendix D3 is the output when the site is in primary zone 7; it prints a note regarding revised depth-area values for Arizona and New Mexico. Appendix D4 is the output when the option to input 12 precipitation values is selected.

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4. Program Execution.

Execution of program PREFRE depends on the computer system being used. Appendix E describes the steps of execution for both the Bureau of Reclamation CYBER mainframe and the IBM PC/AT and compatibles.

Sometimes the site will be very near the boundary between two zones, a situation in which a weighting of calculated frequency values among neighboring zones may provide a more appropriate answer. In these cases, it can be helpful to make more than one run, using the neighboring zone's values. Edit the input file to change the zone number (and other data as needed) and re-run the program.

5. Method of Derivation.

The program follows procedures outlined in NOAA Atlas 2 to derive the precipitation frequency values. The 2-year and 100-year input figures for 6-hour and 24-hour durations are used to derive these same return frequency values for 1-, 2-, and 3-hour durations. The relationships among the 6-hour and 24-hour values and the 1-, 2-, and 3-hour values were determined by the NWS and are dependent on the zone in which the site is located. The 12-hour values are derived by taking the midpoint between the 6-hour and 24-hour input values for the 2-year and 100-year return periods. The 5-, 10-, 15-, and 30-minute duration values for 2-year and 100-year events are determined by multiplying the 1-hour values by a set of factors. These factors are dependent on the short-duration zone in which the site is located. It is important to note that the short-duration zones are different from the primary (longer duration) zones. The program then computes the values for the remaining return periods by fitting the precipitation values to a Gumbel distribution. The 2-year values for all durations are first adjusted from a partial duration series (input values) to an annual series. Then the 5-, 10-, 25-, 50-, and 500-year frequency values for all durations are calculated from their respective relationship to the 2-year and 100-year values in a Gumbel distribution. The 2-, 5-, and 10-year values are then converted back to a partial duration series, which correspond to the NOAA Atlas 2 map values. All output values are for point locations.

NOTE: Areal values of precipitation frequency are often needed. Because program PREFRE does not provide this information, it is necessary to follow the procedure found in the appropriate NOAA Atlas 2 volume. When areal values are required for Arizona and New Mexico, use the information found in the 1984 NOAA Technical Memorandum NWS HYDRO-40.

6. Comments.

It was decided in 1975 to change the program from the procedure originally used by the NWS to a more simplified approach using only the four key precipitation values for input. This allows for quicker setup of the input data and facilitates the use of the program. No loss of accuracy in the calculated values occurs as the 2-year 6-hour, 2-year 24-hour, 100-year 6-hour, and 100-year 24-hour maps are the key maps initially derived in the NWS studies. The maps in NOAA Atlas 2 for return periods of 5, 10, 25, and 50 years were derived from the 2- and 100-year maps in the same manner that the PREFRE program computes these values.

In the original program, only one set of national factors was used to determine 5-min to 30-min values from 1-hour values. Papers by Fredrick and Miller and Arkell and Richards presented sets of factors that depended on the location of the site. These values were used for sites west of the 105th meridian; the old factors were retained for the Plains states east of the 105th meridian.

The 1975 version of the program allowed the user to specify two zones in the event that the site was near a zonal boundary. The current version does not offer that option because two types of zones (the original long-duration zone and the new short-duration zone) are now required and major revisions to the program would be required to accommodate various combinations of multiple runs. The only way to get runs for two adjacent zones is to edit the input file after the first run (a quick and simple procedure) and execute the program again.

4

7. <u>References</u>.

Arkell, R. E., and F. Richards, "Short Duration Rainfall Relations for the Western United States," Preprint, Conference on Climate and Water Management-A Critical Era and Conference on the Consequences of 1985's Climate, August 4-7, 1986, Asheville, NC, Amer. Meteorol. Soc., Boston, 1986.

Frederick, R. H., and J. F. Miller, "Short Duration Rainfall Frequency Relations for California," Preprint, Third Conference on Hydrometeorology, August 20-24, 1979, Bogota, Colombia, Amer. Meteorol. Soc., Boston, 1979.

Miller, J. F., R. H. Frederick, and R. J. Tracy, "NOAA Atlas 2 - Precipitation-Frequency Atlas of the Western United State," 11 volumes, National Weather Service, National Oceanic and Atmospheric Administration, United States Department of Commerce, Silver Spring, Maryland, 1973.

Zehr, R. M., and V. A. Myers, "Depth-Area Ratios in the Semi-Arid Southwest United States," NOAA Technical Memorandum NWS HYDRO-40, Office of Hydrology, National Weather Service, National Oceanic and Atmospheric Administration, United States Department of Commerce, Silver Spring, Maryland, August 1984.

APPENDIX A

Primary zones, used to calculate precipitation for 1 to 24 hr durations. Zone properties are identical to those in NOAA Atlas 2, but zone numbers may differ.

APPENDIX B
Short-duration zones, used to calculate 5 to 30 min durations.

APPENDIX C1

INPUT FORMAT - FOUR PRECIPITATION VALUES

Line 1:
Field 1. Title of study or site name, up to 32 characters

Line 2 (fields separated by blanks or commas):
Field 1. Primary zone number (appendix A)
Field 2. Short-duration zone number (appendix B) *
Field 3. Latitude, degrees and decimals (or 0)
Field 4. Longitude, degrees and decimals (or 0)
Field 5. Elevation (or 0)
Field 6. 0 (number zero)

Line 3 (fields separated by blanks or commas):
Field 1. 2-yr 6-hr precipitation value from NOAA Atlas 2
Field 2. 100-yr 6-hr precipitation value
Field 3. 2-yr 24-hr precipitation value
Field 4. 100-yr 24-hr precipitation value

Line 4 (optional):
Field 1. ENDRUN (alpha characters)

NOTE: Actual latitude and longitude values are required for sites in primary zones 3, 9, and 11, and elevation data are required for sites in primary zones 1, 2, and 6. For other primary zones, enter either zeroes or the latitude, longitude, and elevation values. Elevation may be entered in meters, if precipitation is also metric.

* Short-duration zones 12 through 15 are all for the Southern Pacific Coast. Zone 12 is for sites with elevation greater than 700 ft. Zone 13 is for sites with elevation between 500 and 700 ft. Zone 14 is for sites with elevation less than 500 ft. Zone 15 represents an average of all elevations within the boundaries of the Southern Pacific Coast.

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APPENDIX C2

INPUT FORMAT - TWELVE PRECIPITATION VALUES

Line 1: same as for four precipitation values

Line 2:
Fields 1 through 5: same as for four precipitation values
Field 6. 2

Line 3:
Field 1. 2-yr 6-hr precipitation value from NOAA Atlas 2
Field 2. 5-yr 6-hr precipitation value
Field 3. 10-yr 6-hr precipitation value
Field 4. 25-yr 6-hr precipitation value
Field 5. 50-yr 6-hr precipitation value
Field 6. 100-yr 6-hr precipitation value
Field 7. 2-yr 24-hr precipitation value
Field 8. 5-yr 24-hr precipitation value
Field 9. 10-yr 24-hr precipitation value
Field 10. 25-yr 24-hr precipitation value
Field 11. 50-yr 24-hr precipitation value
Field 12. 100-yr 24-hr precipitation value

Line 4 (optional):
Field 1. ENDRUN (alpha characters)

APPENDIX C3

SAMPLE INPUT - FOUR PRECIPITATION VALUES

Fields separated by blanks

```
QUARTZ HILL, COLORADO
6 7 39.80 105.52 8900 0
1.19 2.05 1.78 4.21
ENDRUN
```

Fields separated by commas

```
LEADVILLE, COLORADO
7,6,39.27,106.31,0,0
.79,1.85,1.00,2.79
ENDRUN
```

SAMPLE INPUT - 12 PRECIPITATION VALUES

```
KUTCH (NW), COLORADO
7 6 39.00 104.00 6100 2
1.04 1.20 2.00 2.25 2.40 2.50 1.39 1.75 1.90 2.25 2.60 3.30
ENDRUN
```

APPENDIX D1

SAMPLE OUTPUT – CYBER

REVISED JUNE 1988 TO UPDATE COMPUTATION OF SHORT-DURATION VALUES

PRECIPITATION FREQUENCY VALUES FOR QUARTZ HILL, COLORADO
PRIMARY ZONE NO.= 6 SHORT-DURATION ZONE NO.= 7
LATITUDE 39.80N LONGITUDE 105.52W ELEVATION 8900 FEET

POINT VALUES

	RETURN PERIOD							
DURATION	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	500-YR	
5-MIN	.26	.34	.39	.47	.53	.59	.73	5-MIN
10-MIN	.40	.53	.62	.74	.84	.93	1.16	10-MIN
15-MIN	.48	.66	.78	.94	1.07	1.20	1.49	15-MIN
30-MIN	.65	.90	1.06	1.29	1.47	1.65	2.05	30-MIN
1-HR	.78	1.09	1.30	1.59	1.81	2.03	2.54	1-HR
2-HR	.92	1.26	1.50	1.82	2.06	2.31	2.88	2-HR
3-HR	1.03	1.39	1.64	1.99	2.25	2.52	3.13	3-HR
6-HR	1.19	1.60	1.87	2.26	2.55	2.85	3.53	6-HR
12-HR	1.49	1.98	2.32	2.80	3.16	3.53	4.37	12-HR
24-HR	1.78	2.37	2.78	3.34	3.78	4.21	5.21	24-HR

INPUT DATA

PROJECT NAME-QUARTZ HILL, COLORADO
ZONE- 6 SHORT-DURATION ZONE- 7
LATITUDE= 39.80 LONGITUDE= 105.52 ELEVATION= 8900
2-YR, 6-HR PCPN- 1.19 100-YR, 6-HR PCPN= 2.85
2-YR, 24-HR PCPN= 1.78 100-YR, 24-HR PCPN= 4.21

```
AAAAAAAAAAAAAA
A            A
A END OF RUN A
A            A
AAAAAAAAAAAAAA
```

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APPENDIX D2

SAMPLE OUTPUT - PC

*** O U T P U T   D A T A ***

REVISED JUNE 1988 TO UPDATE COMPUTATION OF SHORT-DURATION VALUES

PRECIPITATION FREQUENCY VALUES FOR QUARTZ HILL, COLORADO
PRIMARY ZONE NUMBER= 6
SHORT-DURATION ZONE NUMBER= 7

LATITUDE 39.80N LONGITUDE 105.52W ELEVATION 8900 FEET

POINT VALUES

DURATION	RETURN PERIOD 2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	500-YR	
5-MIN	.26	.34	.39	.47	.53	.59	.73	5-MIN
10-MIN	.40	.53	.62	.74	.84	.93	1.16	10-MIN
15-MIN	.48	.66	.78	.94	1.07	1.20	1.49	15-MIN
30-MIN	.65	.90	1.06	1.29	1.47	1.65	2.05	30-MIN
1-HR	.78	1.09	1.30	1.59	1.81	2.03	2.54	1-HR
2-HR	.92	1.26	1.50	1.82	2.06	2.31	2.88	2-HR
3-HR	1.03	1.39	1.64	1.99	2.25	2.52	3.13	3-HR
6-HR	1.19	1.60	1.87	2.26	2.55	2.85	3.53	6-HR
12-HR	1.49	1.98	2.32	2.80	3.16	3.53	4.37	12-HR
24-HR	1.78	2.37	2.78	3.34	3.78	4.21	5.21	24-HR

INPUT DATA

PROJECT NAME=QUARTZ HILL, COLORADO
ZONE= 6 SHORT-DURATION ZONE= 7
LATITUDE= 39.80 LONGITUDE= 105.52 ELEVATION= 8900
2-YR, 6-HR PCPN= 1.19 100-YR, 6-HR PCPN= 2.85
2-YR, 24-HR PCPN= 1.78 100-YR, 24-HR PCPN= 4.21

* * * * E N D OF R U N * * * *

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APPENDIX D3

SAMPLE OUTPUT - PC (PRIMARY ZONE 7)

*** O U T P U T D A T A ***

REVISED JUNE 1988 TO UPDATE COMPUTATION OF SHORT-DURATION VALUES

PRECIPITATION FREQUENCY VALUES FOR LEADVILLE, COLORADO
PRIMARY ZONE NUMBER= 7
SHORT-DURATION ZONE NUMBER= 6

LATITUDE 39.27N LONGITUDE 106.31W ELEVATION 10200 FEET

POINT VALUES

	RETURN PERIOD							
DURATION	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	500-YR	
5-MIN	.20	.26	.30	.36	.41	.45	.56	5-MIN
10-MIN	.31	.41	.47	.57	.64	.71	.88	10-MIN
15-MIN	.37	.50	.58	.70	.79	.88	1.09	15-MIN
30-MIN	.48	.64	.75	.91	1.03	1.15	1.43	30-MIN
1-HR	.58	.78	.92	1.12	1.27	1.42	1.77	1-HR
2-HR	.65	.87	1.03	1.24	1.40	1.57	1.94	2-HR
3-HR	.70	.93	1.09	1.32	1.49	1.66	2.06	3-HR
6-HR	.79	1.05	1.22	1.47	1.66	1.85	2.29	6-HR
12-HR	.89	1.25	1.49	1.81	2.07	2.32	2.90	12-HR
24-HR	1.00	1.45	1.75	2.16	2.48	2.79	3.52	24-HR

* IF YOUR SITE IS IN ARIZONA OR NEW MEXICO, PLEASE CONSULT THE FOLLOWING PAPER FOR REVISED DEPTH-AREA VALUES:
DEPTH-AREA RATIOS IN THE SEMI-ARID SOUTHWEST UNITED STATES
NOAA TECHNICAL MEMORANDUM NWS HYDRO-40
ZEHR AND MYERS
AUGUST 1984

INPUT DATA

PROJECT NAME=LEADVILLE, COLORADO
ZONE= 7 SHORT-DURATION ZONE= 6
LATITUDE= 39.27 LONGITUDE= 106.31 ELEVATION=10200
2-YR, 6-HR PCPN= .79 100-YR, 6-HR PCPN= 1.85
2-YR, 24-HR PCPN= 1.00 100-YR, 24-HR PCPN= 2.79

* * * * E N D OF R U N * * * *

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APPENDIX D4

SAMPLE OUTPUT - PC (12 PRECIP VALUES)

```
                *** O U T P U T   D A T A ***
 REVISED JUNE 1988 TO UPDATE COMPUTATION OF SHORT-DURATION VALUES

PRECIPITATION FREQUENCY VALUES FOR KUTCH (NW), COLORADO
 PRIMARY ZONE NUMBER=    7
 SHORT-DURATION ZONE NUMBER=    6

   OPTION NUMBER 2 --- INPUT OF 12 PRECIP VALUES
  LATITUDE    39.00N      LONGITUDE   104.00W      ELEVATION    6100 FEET
```

POINT VALUES

DURATION	RETURN PERIOD 2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	500-YR	
5-MIN	.29	.40	.47	.57	.65	.72	.90	5-MIN
10-MIN	.45	.61	.73	.89	1.01	1.13	1.41	10-MIN
15-MIN	.54	.75	.90	1.09	1.25	1.40	1.75	15-MIN
30-MIN	.68	.97	1.16	1.42	1.63	1.83	2.30	30-MIN
1-HR	.82	1.18	1.42	1.75	2.01	2.26	2.84	1-HR
2-HR	.91	1.28	1.53	1.87	2.14	2.40	3.01	2-HR
3-HR	.96	1.34	1.60	1.95	2.22	2.49	3.12	3-HR
6-HR	1.06	1.46	1.73	2.10	2.38	2.67	3.33	6-HR
12-HR	1.17	1.58	1.86	2.25	2.56	2.86	3.55	12-HR
24-HR	1.28	1.71	2.00	2.41	2.73	3.05	3.78	24-HR

```
* IF YOUR SITE IS IN ARIZONA OR NEW MEXICO, PLEASE CONSULT THE
  FOLLOWING PAPER FOR REVISED DEPTH-AREA VALUES:
    DEPTH-AREA RATIOS IN THE SEMI-ARID SOUTHWEST UNITED STATES
    NOAA TECHNICAL MEMORANDUM NWS HYDRO-40
    ZEHR AND MYERS
    AUGUST 1984


              INPUT DATA

PROJECT NAME=KUTCH (NW), COLORADO
ZONE= 7     SHORT-DURATION ZONE= 6
LATITUDE= 39.00      LONGITUDE= 104.00      ELEVATION= 6100
  12-VALUE PRECIPITATION OPTION
 PRECIPITATION VALUE:
 1.04        1.20
 2.00        2.25
 2.40        2.50
 1.39        1.75
 1.90        2.25
 2.60        3.30


          * * * *  E N D   OF   R U N  * * * *
```

14

APPENDIX E

EXECUTION OF PROGRAM PREFRE

CYBER

The following steps are used to execute program PREFRE on the Bureau of Reclamation CYBER mainframe computer:

1. Create an input file, using any convenient name, following the format presented in appendix C. This becomes a permanent file on the CYBER. Purge it when it is no longer needed.

2. Enter OLD,PREFREB [the binary (executable) form]
then GET,INPUT=your input file name
then PREFREB

3. The output information is sent to the screen. It can also be printed; use the procedures appropriate for the hardware available to you.

Personal Computer

PREFRE is the executable version of the program. It may be stored on the hard disk or it may be on a floppy disk. The following steps are used to execute the program on an IBM PC/AT or compatible (a FORTRAN compiler must be available on the particular PC being used):

1. Create an input file, using any convenient name, following the format presented in appendix C. This is a permanent file on the hard disk or floppy disk.

2. For hard disk, enter PREFRE filename1 filename2
(e.g., PREFRE PREIN1 PREOUT1)
For floppy disk, enter A:PREFRE filename1 filename2
(e.g., A:PREFRE A:PREIN1 A:PREOUT1)

Filename1 (including device ID and name extension) is the name of your input file and filename2 (including device ID and name extension) is the name of the file you wish the output information written. Either or both files may be on the hard disk or they may be on a floppy disk in device A. If they are on a floppy disk, the filename must be preceded by A:. The output file will be created by the program. If you fail to enter the file names at this point, the program will prompt you to enter those names. Messages will appear on the screen, but the output data are written to the file.

3. Enter PRINT filename2

15

APPENDIX E (continued)

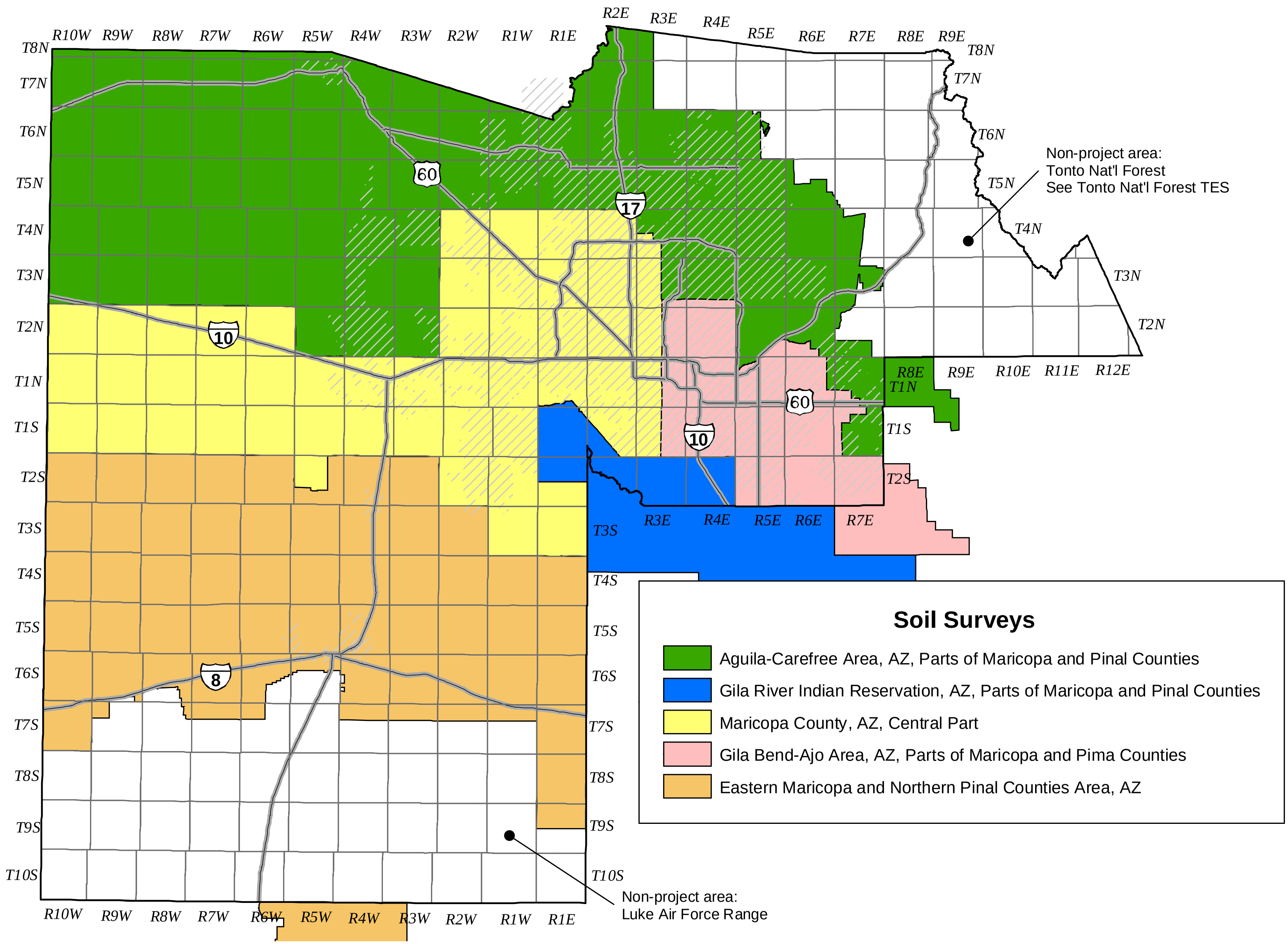
The output data will be listed at the printer. If you directed the output file to be written to the floppy disk (in device A), enter PRINT A:filename2. The output file is also a permanent file on the hard disk or floppy disk.

APPENDIX B: INTENSITY-DURATION-FREQUENCY GRAPH

B.1 Section 1: Intensity-Duration-Frequency Graph

APPENDIX C: LOSS RATE PARAMETER TABLES

C.1 Section 1: General


Soil Surveys
Aguila-Carefree Area, AZ, Parts of Maricopa and Pinal Counties
Gila River Indian Reservation, AZ, Parts of Maricopa and Pinal Counties
Maricopa County, AZ, Central Part
Gila Bend-Ajo Area, AZ, Parts of Maricopa and Pima Counties
Eastern Maricopa and Northern Pinal Counties Area, AZ
Non-project area:
Tonto Nat'l Forest
See Tonto Nat'l Forest TES
Non-project area:
Luke Air Force Range
R10W R9W R8W R7W R6W R5W R4W R3W R2W R1W R1E R2E R3E R4E R5E R6E R7E R8E R9E R10E R11E R12E
T8N T7N T6N T5N T4N T3N T2N T1N T1S T2S T3S T4S T5S T6S T7S T8S T9S T10S
60
17
10
8

APPENDIX C: LOSS RATE PARAMETER TABLES

C.2 Section 2: Aguila-Carefree Soil Survey

APPENDIX C: LOSS RATE PARAMETER TABLES

C.3 Section 3: Maricopa Central Soil Survey

APPENDIX C: LOSS RATE PARAMETER TABLES

C.4 Section 4: Eastern Maricopa/Northern Pinal Soil Survey

APPENDIX D: UNIT HYDROGRAPH

D.1 Section 1: T_c and R Worksheet

APPENDIX D: UNIT HYDROGRAPH

D.2 Section 2: K_n Values

APPENDIX E: DDMSW USERS MANUAL

FLOOD CONTROL DISTRICT
of
MARICOPA COUNTY

Drainage Design Management System

User's Manual

KVL Consultants, Inc.

Table of Contents

1. Introduction

2. General Features

3. File

4. Hydrology

Table of Contents

1. Introduction

System Overview

The Drainage Design Management System for Windows (DDMSW) has been written to facilitate data management and computational procedures required for drainage analysis in Maricopa County. This manual serves as a guide in the use of the program and is intended to be used in conjunction with the County's Drainage Design Manuals.

The program is written in Microsoft Visual FoxPro and currently includes modules for File Management, Hydrology and Utilities. Future versions will include modules for Hydraulics and GIS (Geographic Information System) integration.

Unlike the former DDMS which was DOS based and stored data in separate ASCII files, DDMSW is a relational Database that manages multiple projects from one single location. DDMSW is a multi-tasking window based application which enables the user to open several 'windows' simultaneously. New features include pull-down menus, user-friendly forms which the user can arrange on the desktop, and windows editing tools to facilitate data entry. DDMSW utilizes a relational Database that includes Tables for data entry and editing. Each Table appears as a separate '.dbf' file on disk. The Tables are related to each other based on the key field 'Project Id' which is established when starting a new project. Running models is automated from a menu and the data for running the models is extracted from the various Tables in the Database.

Basic Database Terminology

The application stores data (values) in a relational Database. This data is organized into *tables, fields*, and *records* to make it more meaningful. For example, 01 by itself is meaningless. However, in a table called 'Basins', in a field called 'BasinId', in a record corresponding to 'EXAMPLE1', we now understand that 01 is a major basin in project EXAMPLE1.

A *table* is a grouping of data. The data is dynamic because it can be modified, deleted, added to, and so on. Here is an example of a table:

Table: Basins

ProjectID	BasinID	Description	Sort
EXAMPLE 1	01	Major Basin 01	10
EXAMPLE 1	02	Major Basin 02	20

A table is composed of one or more *fields*. In the example, the fields are ProjectID, BasinID, Description, and Sort. Fields are similar to columns in a spreadsheet. All fields in a table have the same format (eg. text of maximum 70 characters, numeric 12 places with 2 decimals) and they share the same characteristics (eg. they are different descriptions).

A table also consists of one or more *records*. Records are similar to rows in a spreadsheet. In the example, "KVLTEST1, 01, Major Basin 01, 10' compose one record in the table 'Basins'. The example shows a total of two records and four fields.

In DDMSW, the Database is composed of numerous tables which organize and store information. These tables are linked by one common *key field*, **projectid** which identifies the project the records are associated with.

Program Installation

DDMSW

The software used in DDMSW includes:

DDMSW	Compiled application
HEC-1	Most recent HEC-1 with modifications
Prefre	Rainfall model
MCUHP1	County's DOS program
MCUHP2	County's DOS program
Rational	County's DOS program
PFE	Text editor (Programmer's File Editor)
Acrobat Reader	PDF file reader

All required software for DDMSW is contained on the CD provided with this application.

Insert the DDMSW CD in the CD drive (here denoted as X). Run X:\DDMSW\Setup from the RUN command (substitute your CD drive letter for X). Follow the instructions on the screen.

The user can choose the program's location, but assuming C:\DDMSW, the following directory structure will be created:

C:\DDMSW	Program files
C:\DDMSW\Adobe	Adobe Acrobat installation files
C:\DDMSW\Backup	Directory for archiving data
C:\DDMSW\Data	Data Files
C:\DDMSW\Help	Help files
C:\DDMSW\ModlRuns	Default directory for model runs
C:\DDMSW\Models	Model programs
C:\DDMSW \Reports	Reports

The procedure will notify the user when the DDMSW installation is complete.

Adobe Acrobat Reader

This manual and all help files require Adobe Acrobat Reader to view and print the files. If Adobe Acrobat Reader is not currently installed on your computer, then it will be necessary to install the program. The latest version can be downloaded from Adobe's website at WWW.Adobe.Com. Alternatively, a copy of Adobe Reader is included with this application. To install, click on the executable file in the 'Abobe' subdirectory of DDMSW and follow the instructions on the screen.

Windows Regional Settings

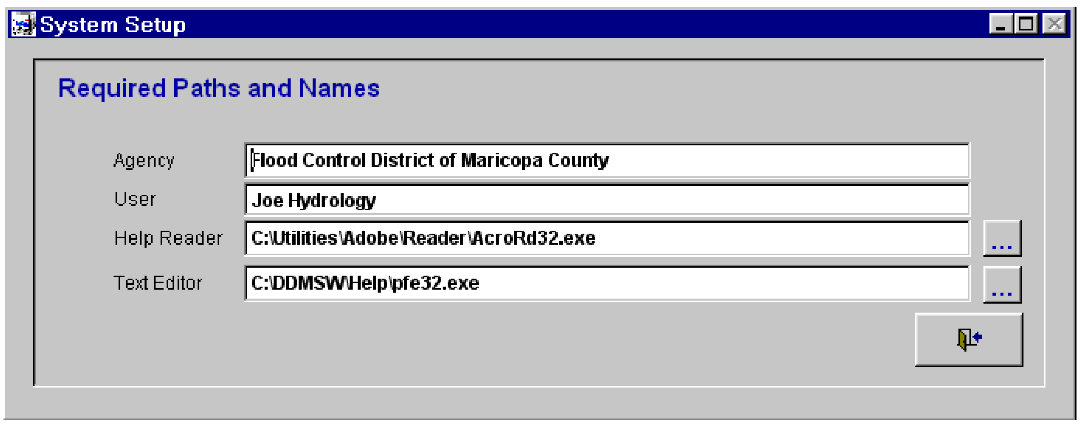
To ensure that printed reports contain the necessary number of decimal places, it is necessary to modify the regional Settings in the Windows Control Panel as follows:

Open Regional Settings (found in Control Panel) and click on 'Number'. Change "No of digits after decimal" to 5 and then click Apply.

Starting the Software

DDMSW is started by clicking on Start\Programs\DDMSW\FCDMC\DDMSW.exe (provided this was where the software was installed). The program can also be started by double clicking on DDMSW.exe in the folder where the software is installed.

When the software is first started, it is necessary to edit File\Setup to establish system settings.



2. General Features

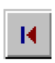
Main Menu

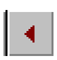
The Main Menu is the center of the application. This is the screen which is displayed when the user starts the application. This is also the screen the user is always returned to after closing a submenu or form.

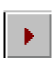
Specific actions can be accessed through the pull-down menus shown on the Main Menu bar. This manual will explain the functions available on each menu and will describe the individual elements shown on data entry screens.

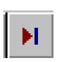
Standard Buttons

There is a toolbar of standard buttons, which is identical on each data entry screen.

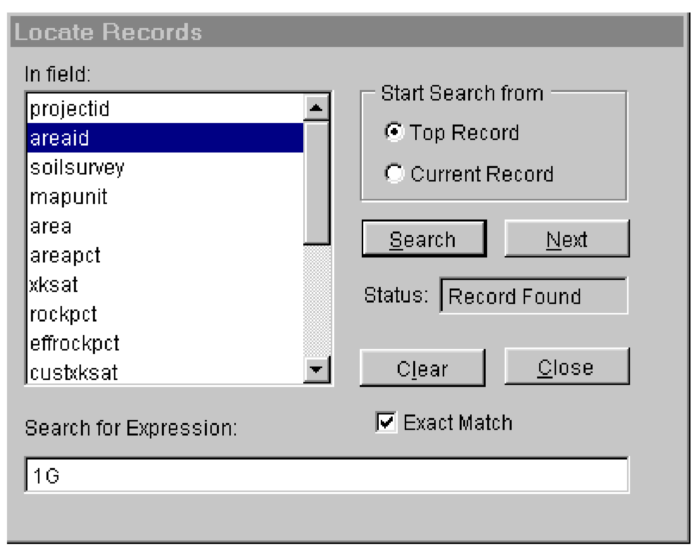

Goes to the first record in the table.


Moves to the previous record.

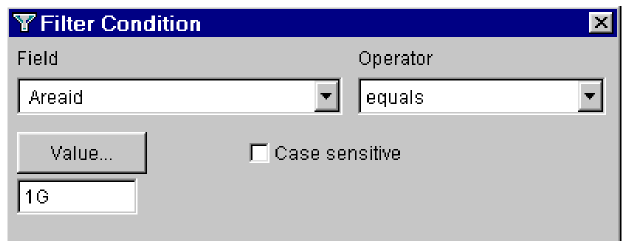

Moves to the next record.


Goes to the last record in the table.


Locates records based on a specified search criterion. Highlight the field to search and type the search expression.



Selects a subset of records according to user specifications. A *filter* consists of one or more conditions which compares a field to a value using an *operator*. Multiple conditions can be combined together with a *connector*. To create a filter, select the 'Add' button. The following 'Filter Condition' dialog box appears.



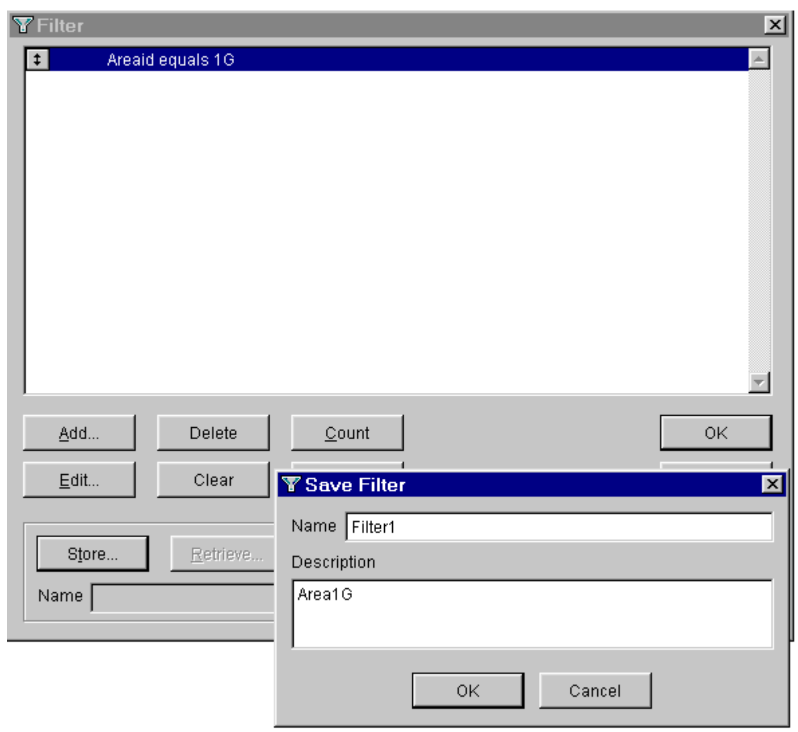
Select a field and an operator, enter a value and click 'OK'. (For alphanumeric fields, select a value from the pull down list).

On the 'Filter' dialog box, the user has the following options:

i) Click 'OK' to execute the filter and view the subset of records.

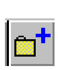
ii) Select 'Add' again to add other condition to the filter.

iii) Select 'Store' to save the filter for future use. This is useful for commonly used filters.

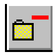


To use a previously saved filter, select 'Retrieve'. Highlight the filter on the 'Select Filter' dialog box and click 'OK'. Click 'OK' on the 'Filter' dialog box to execute the command.

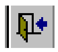
To edit a filter, retrieve it from the list, and choose 'Edit' or double-click on the condition. After changing any of the items that make up the condition, select 'Store' and 'OK' to save the changes.



Adds a new record.



Marks the current record for deletion. Marked records are physically deleted from disk when the Table is packed. The record will no longer appear but still exists until the Table is packed.



Closes the current form and returns the user to the Main Menu or previous form. Any changes made to the record are saved. Pressing [Esc] will also close the form and return the user to the previous screen. However, changes to the current record may not take effect.

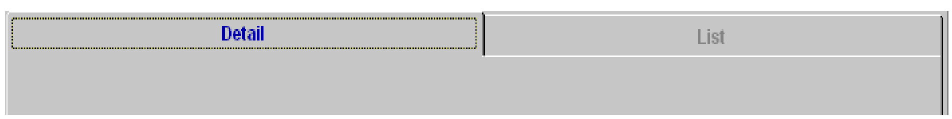
Edit Menu

The Edit menu is available to the user during data entry or editing. The menu comprises the following functions. Some or all may be available depending on the action currently being executed.

Undo	Undo the last change made to a field.
Cut	Cut out (move) the highlighted text to the clipboard.
Copy	Copy the highlighted text to the clipboard.
Paste	Paste the text from the clipboard into the current field.

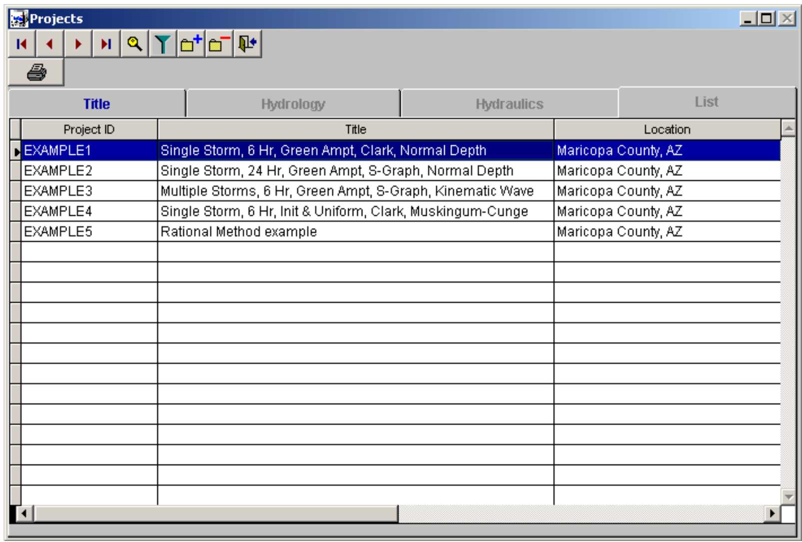
Forms

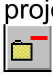
Some forms are composed of several tabs to view data. Click on the tab for the appropriate view.

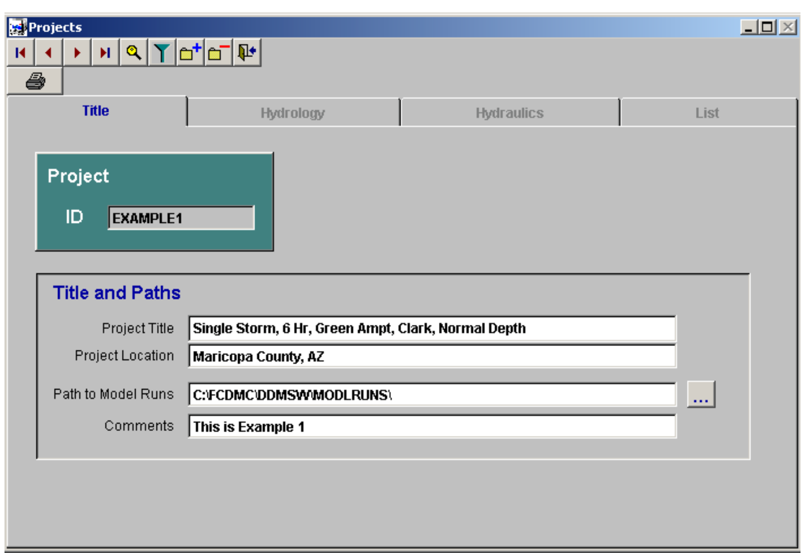


3. File

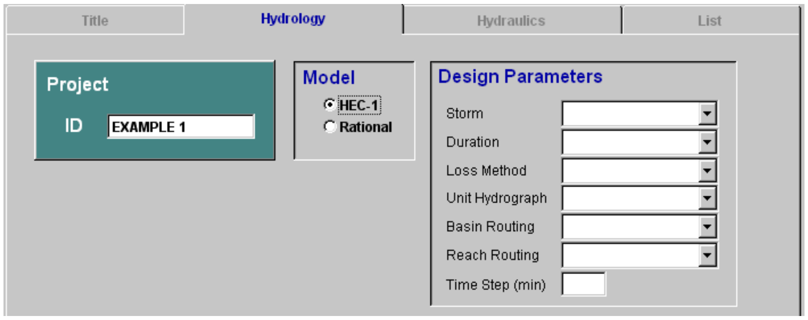
Select Project



The selection of Select Project from the File Menu is used to select, edit or create a new project. To add a new Project, click on the  button. To delete a Project, click on the  button. When adding a new Project, it is necessary to fill in the Project ID, Title and the path to the model runs on the Title form. The Project ID will be used throughout the application for all files that refer to this project. Model runs for all projects will have the same naming convention. It is therefore necessary to **establish separate folders for each project's model runs.** Create the folder using Windows Explorer and then use the ... button to locate the folder.

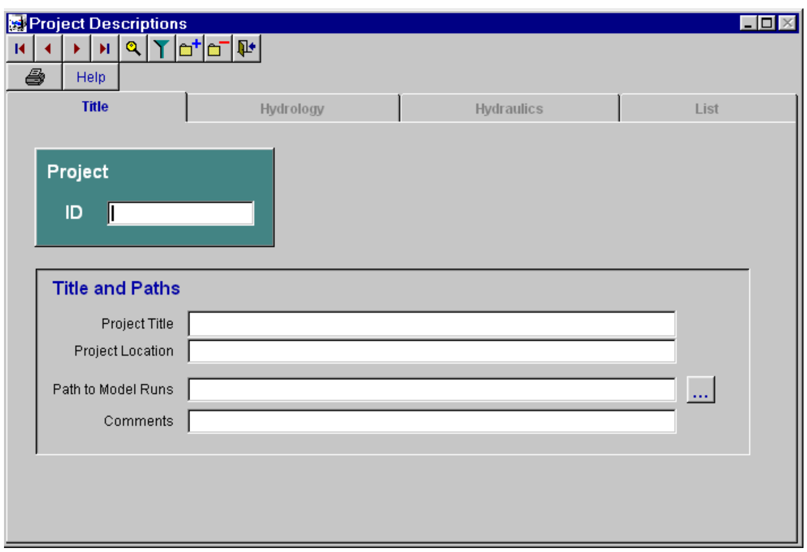


When finished with the Title data, enter the Hydrology default data. The Hydrology view varies depending on which model is selected.



All data on this form can be modified. Once the form has been closed, the Project ID cannot be edited. If it is necessary to change the Project ID, "*Rename"* the project in Management. If default data is changed, make sure that the appropriate data on other forms is also modified. For example, if the reach routing method is changed from "Normal Depth" to "Kinematic Wave", it will be necessary to modify the routing data.

New Project



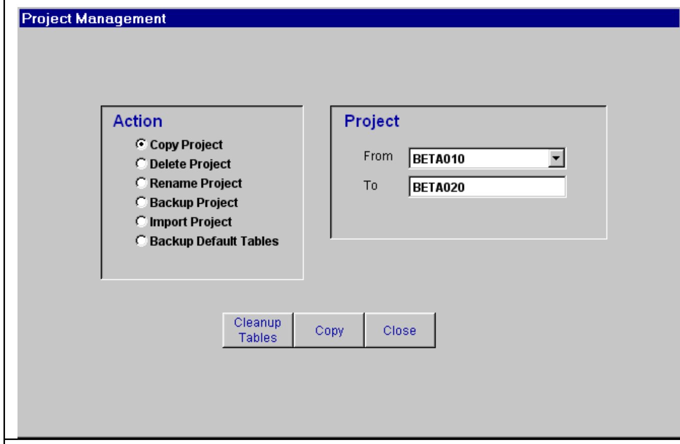
The selection of New Project from the File Menu is used to create a new project. When creating a new project, it is necessary to fill in the Project ID, Title and the path to the model runs on the Title page. All other features are the same as Select Project when adding a new record (see previous section). When selecting New Project from the menu, a new record is automatically added to the projects table.

Management

The Management menu offers the following functions:

<table>
<tr>
<td>

Copy a Project

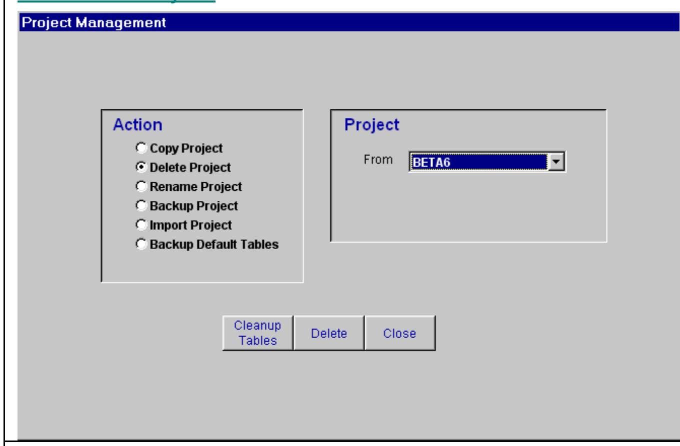


</td>
<td>

Creates a new project and copies all records in the Tables to the new project. Model files are not copied. Select the current project from the drop-down list and enter the name of the new project in the 'To' field. Click the 'Copy' button.

</td>
</tr>
<tr>
<td>

Delete a Project

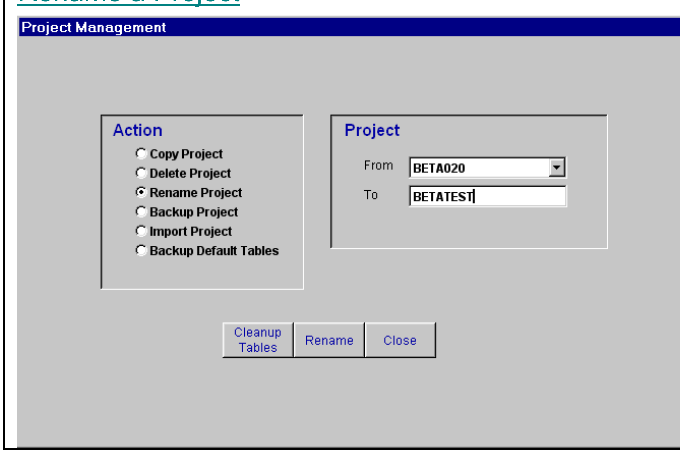


</td>
<td>

Select the project from the drop-down list and click the 'Delete' button. All relevant records in the Tables are deleted. It is recommended to pack the Database to erase the deleted records from disk (see 'Utilities').

</td>
</tr>
<tr>
<td>

Rename a Project

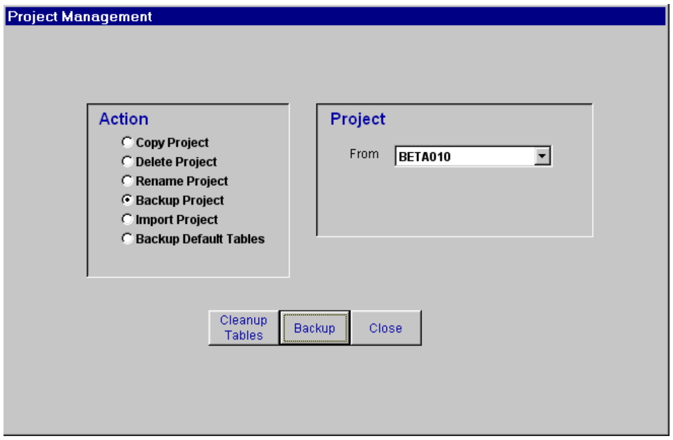


</td>
<td>

Use this function to change the name of a project. Select the project from the drop-down list and enter the new project name in the 'To' field. Click the 'Rename' button.

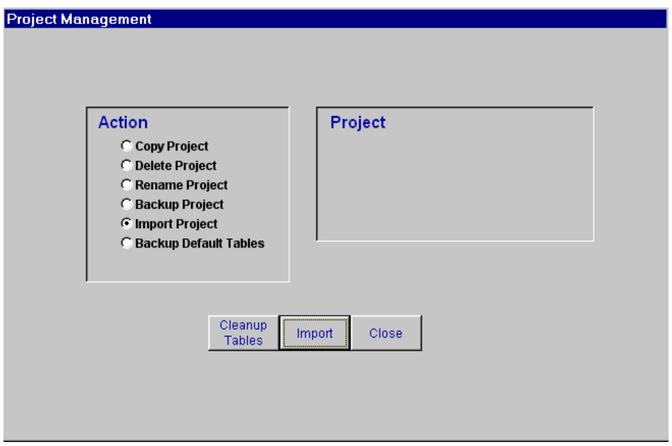
</td>
</tr>
</table>

Backup Project



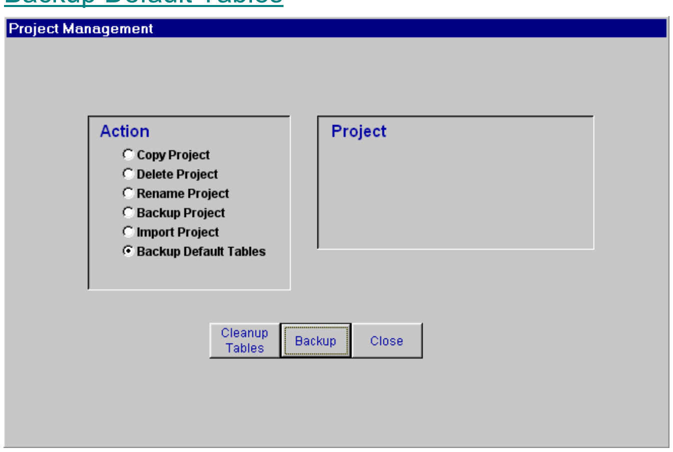
Use this option to backup project data to one '*zipped'* file. The backup file is saved to the backup subdirectory and has the project name with a '.zip' extension. This feature is useful when a DDMSW project needs to be used on a different computer.

Import Project



When the 'Import' button is clicked, a dialogue box appears for the user to select a project backup file (.zip extension). If the project already exists in the Database, a message appears to warn the user that all data in the current project will be deleted and replaced with the imported data. The user has the choice to continue or cancel. This feature allows users to import a DDMSW project (zip format) generated by DDMSW's Backup Project feature.

Backup Default Tables



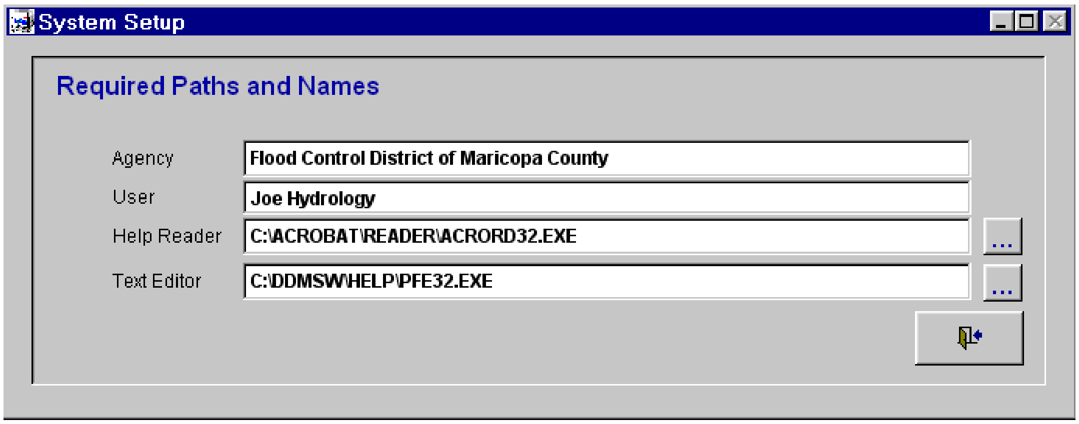
This option backs up the default data to a file 'defaults.zip' in the backup subdirectory. To restore the default data, copy the defaults.zip file to the data subdirectory and unzip the file.

This form can also be used to get rid of "orphans" in DDMSW by clicking on "Cleanup Tables". Orphans records are records in a Table that do not belong to a Project ID.

Setup

The selection of Setup from the File Menu is used to edit System settings. These include:

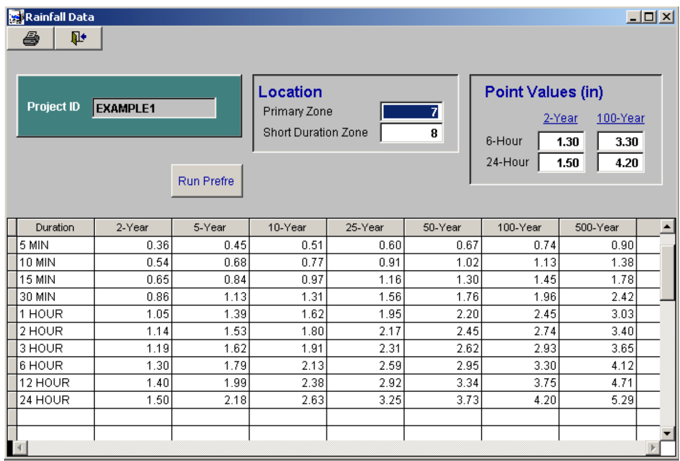
- Agency name
- User's name
- Path and file name to Adobe Acrobat Reader which is necessary to view help files.
- Path and file name for a text editor (PFE is supplied with the application and is located in the help subdirectory of DDMSW).



4. Hydrology

Rainfall

Prefre

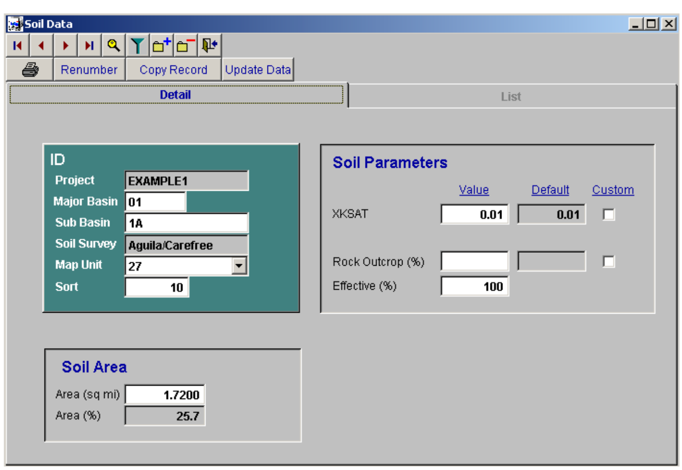


The selection of Prefre from the Hydrology\Rainfall menu is used to edit data required for running the Prefre model and to run the model. The Prefre program computes the rainfall depths for the durations and return periods shown on the form. The user must enter appropriate values for all fields shown on the form. The selection of data can be obtained from the County's Drainage Design Manuals. For Maricopa County, the Primary Zone is always 7 and the Short Duration Zone is always 8.

Soils

Data

The selection of Data from the Hydrology\Soil menu is used to add or edit soil data required for the Sub Basins.

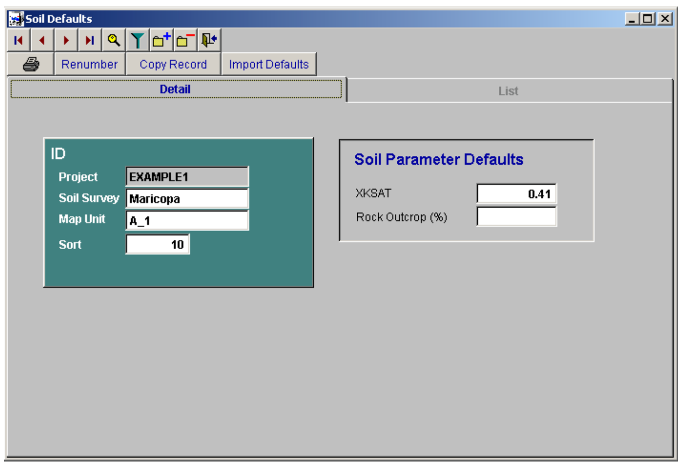


The important values to enter are the Sub Basin ID which must match a Sub Basin ID in the Sub Basins Table, a Map Unit code, which will come from the Soil Defaults and the area for this soil. All of the remaining values can be calculated using the Update Data button.

There are three columns in the Soil Parameters section of the form. The first column, Value, is the value that will be used in the modeling analysis. The second column, Default, is the value calculated based on soil default values. The third column, Custom, ensures that a user entered value will not be overwritten with the calculated value when updating data.

Defaults

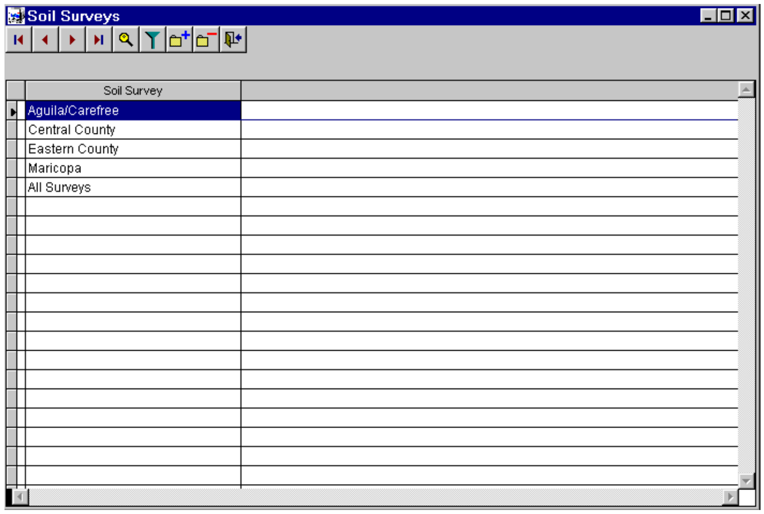
The selection of Defaults from the Hydrology\Soil menu is used to add or edit soil defaults required for the Sub Basins.



It is important to enter appropriate values for all fields as soil calculations for the entire project will be based on these values. When first entering this form for the project, if soil default data does not exist, the County's default data is loaded into the system.

Soil Surveys

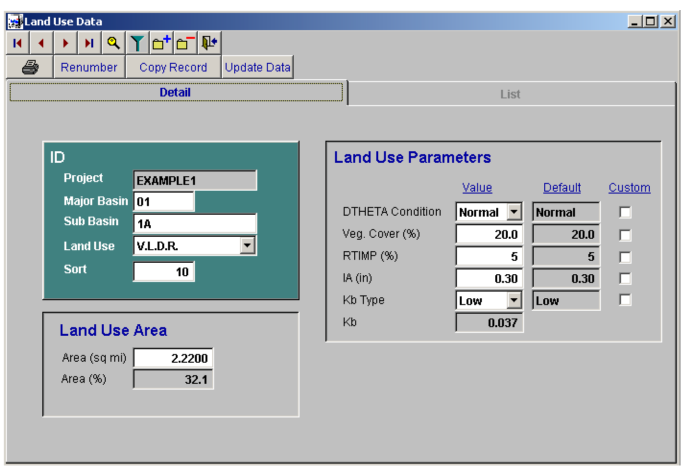
The selection of Soil Surveys from the Hydrology\Soil menu is used to add or edit soil surveys. This data is used to filter data on the Soil Data form.



Land Use

Data

The selection of Data from the Hydrology\Land Use menu is used to add or edit land use data required for the Sub Basins. The form is different for HEC-1 and Rational models.

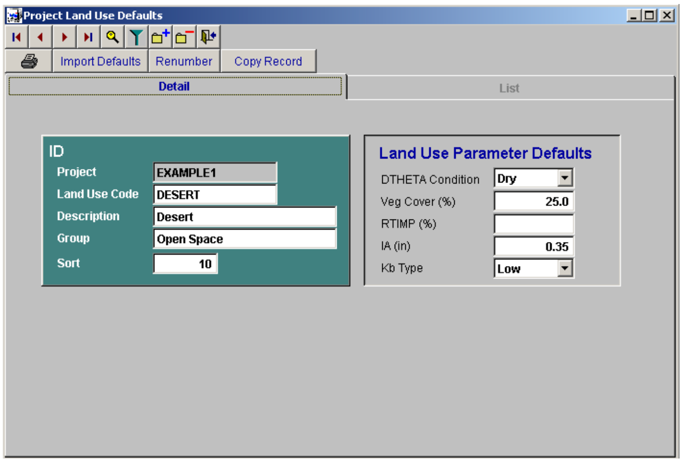


The important values to enter are the Sub Basin ID which must match a Sub Basin ID in the Sub Basins Table, a land use code, which will come from the Land Use Defaults and the area for this land use. All of the remaining values can be calculated using the *"Update Data"* button.

There are three columns in the Land Use Parameters section of the form. The first column, 'Value', is the value that will be used in the modeling analysis. The second column, 'Default', is the calculated value based on the land use default values. The third column, 'Custom', ensures that a user-entered value will not be overwritten with the calculated default value when updating data.

Defaults

The selection of Defaults from the Hydrology\Land Use menu is used to add or edit land use defaults required for the Sub Basins. The form is different for HEC-1 and Rational models.

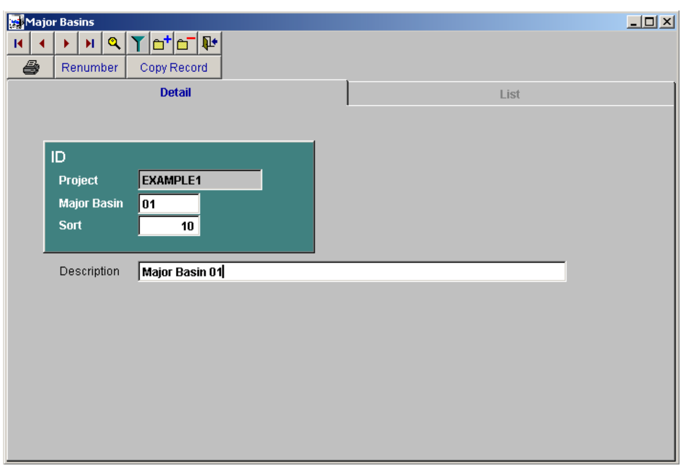


It is important to enter appropriate values for all fields as land use calculations for the entire project will be based on these values.

Basins

Major Basins

The selection of Major Basins from the Hydrology\Basins menu is used to add a new or edit an existing Major Basin.



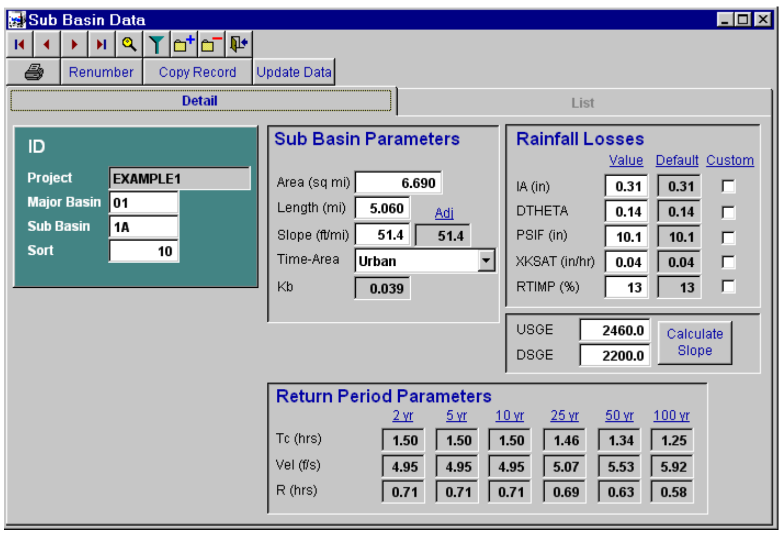
Major Basins within a project are drainage basins that generally have a major outfall. Major Basins will have separate HEC-1 input files and their ID is designated by a **__two character field__**. Single digit numbers must therefore be preceded by a zero. Within a project, the first Major Basin ID is designated as "01", the second as "02" etc. until Basin "99" is reached. If the number of basins exceeds ninety-nine, a new project must be started. It is necessary to have at least one Major Basin (01) in a project.

Sub Basins

The selection of Sub Basins from the Hydrology\Basins menu is used to add a new or edit an existing Sub Basin. The form is different for HEC-1 and Rational models.

Sub Basins are drainage areas within a Major Basin. Sub Basin IDs are designated by a six character field **and must be unique within a project**. The preferred practice will be to designate the first two characters with the Major Basin ID and the remaining four characters in some systematic order.

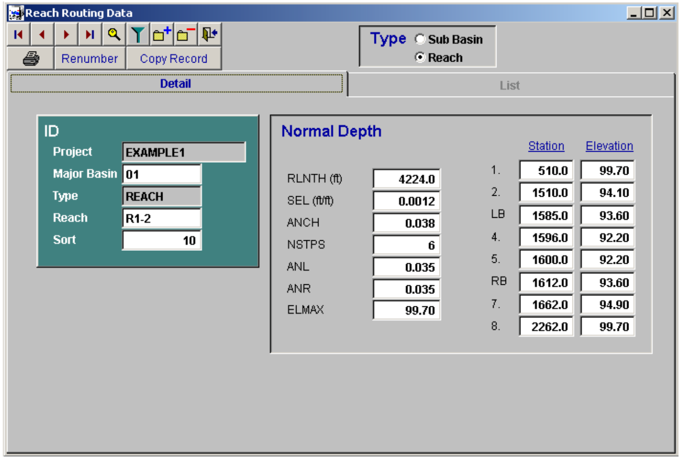
Fields appearing on the Sub Basin form will vary depending on the defaults established in the project setup. For example if the Clark Unit Hydrograph is selected as the default, then the parameters for an S-Graph will not be available. There are three columns in the Rainfall Losses section of the form. The first column, 'Value', is the value that will be used in the modeling analysis. The second column, 'Default', is the calculated value based on the default values in Land Use and Soils. The third column, 'Custom', ensures that a user entered value will not be overwritten with the calculated value when updating data.



Update Data recalculates all values based the procedures established in the County's Drainage Design Manuals. Values with the Custom box checked will not be updated.

Routing

The selection of Routing from the Hydrology\Basins menu is used to add a new or edit existing Sub Basin or Reach routing data.

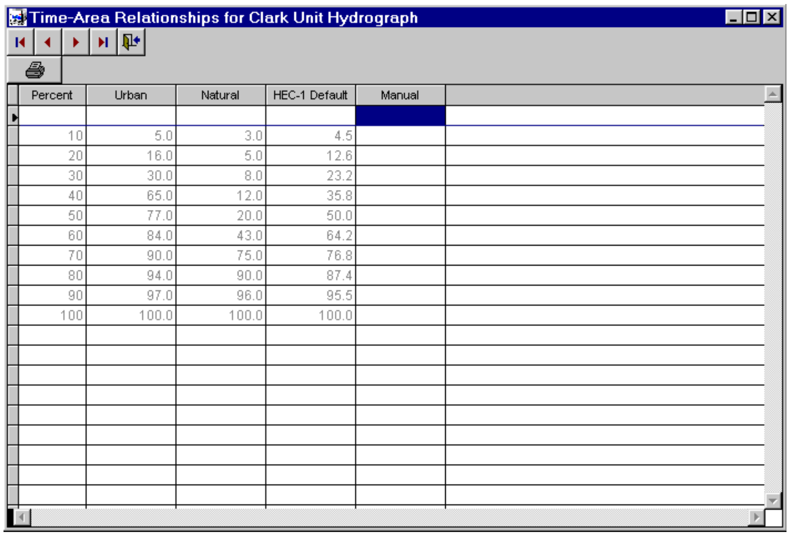


If Sub Basin is checked in the *"Type"* box at the top of the form, then Sub Basin routing data is available for editing. Likewise, if Reach is selected, then reach routing is available for editing. The Sub Basin ID must be unique and must match an ID in the Sub Basin data. The Reach ID must be unique within a project. In Maricopa County, routing is used only for Reach Routing. Basin routing is seldom used.

Distributions

Time-Area

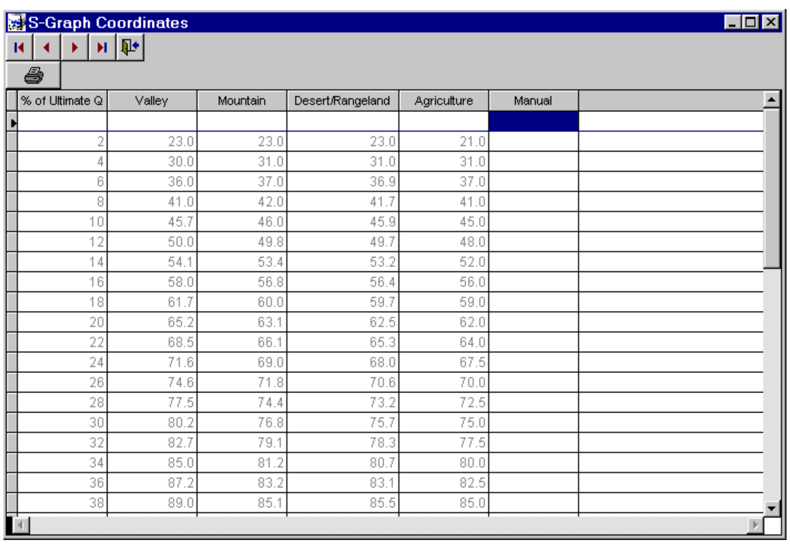
The selection of Time-Area from the Hydrology\Distributions menu is used to edit Time-Area distributions used for the Clark Unit Hydrograph. Only data in the *"Manual"* column can be edited. Time-Area is only available if "Clark" is selected as the default unit hydrograph.


Time-Area Relationships for Clark Unit Hydrograph

Percent	Urban	Natural	HEC-1 Default	Manual
10	5.0	3.0	4.5	
20	16.0	5.0	12.6	
30	30.0	8.0	23.2	
40	65.0	12.0	35.8	
50	77.0	20.0	50.0	
60	84.0	43.0	64.2	
70	90.0	75.0	76.8	
80	94.0	90.0	87.4	
90	97.0	96.0	95.5	
100	100.0	100.0	100.0	

S-Graph

The selection of S-Graph from the Hydrology\Distributions menu is used to edit S-Graph distributions used for the S-Graph Unit Hydrograph. Only data in the *"Manual"* column can be edited. S-Graph is only available if "S-Graph" is selected as the default unit hydrograph.


S-Graph Coordinates

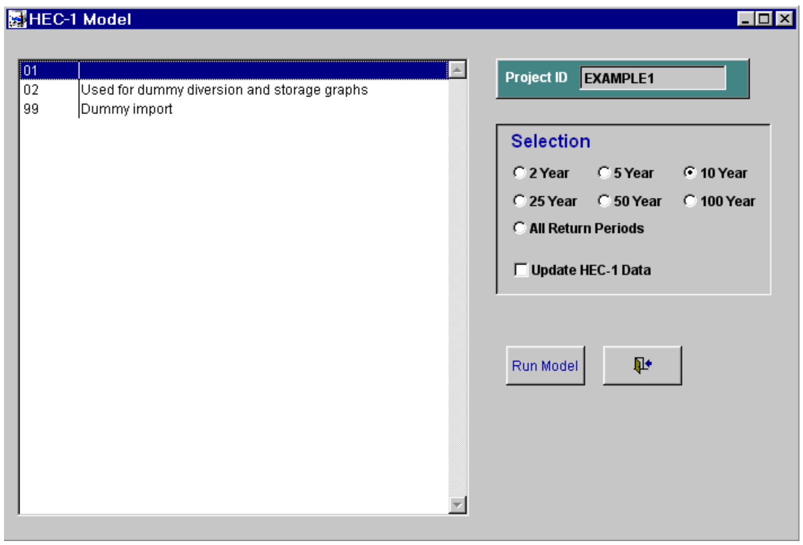
% of Ultimate Q	Valley	Mountain	Desert/Rangeland	Agriculture	Manual
2	23.0	23.0	23.0	21.0	
4	30.0	31.0	31.0	31.0	
6	36.0	37.0	36.9	37.0	
8	41.0	42.0	41.7	41.0	
10	45.7	46.0	45.9	45.0	
12	50.0	49.8	49.7	48.0	
14	54.1	53.4	53.2	52.0	
16	58.0	56.8	56.4	56.0	
18	61.7	60.0	59.7	59.0	
20	65.2	63.1	62.5	62.0	
22	68.5	66.1	65.3	64.0	
24	71.6	69.0	68.0	67.5	
26	74.6	71.8	70.6	70.0	
28	77.5	74.4	73.2	72.5	
30	80.2	76.8	75.7	75.0	
32	82.7	79.1	78.3	77.5	
34	85.0	81.2	80.7	80.0	
36	87.2	83.2	83.1	82.5	
38	89.0	85.1	85.5	85.0	

HEC-1

Run HEC-1

This function runs the HEC-1 model for selected Major Basins within a project.

HEC-1 can be run for major basins where supporting data exists and also for an imported HEC-1 input file where no supporting data exists. To run a HEC-1 model that has been developed elsewhere, simply establish either a new project or a new major basin within an existing project and import the HEC-1 input file in Edit HEC-1 Data. It is also necessary to establish whether the model uses Multiple or Single storms in "Select Project". When running the model, do not check "Update Data" as this feature will not be available.



Highlight the Major Basin to model and select the appropriate return period. The results of the model run(s) will be place in a user defined directory established in the project defaults. The resultant file names will be distinguished by the Major Basin and return period being modeled.

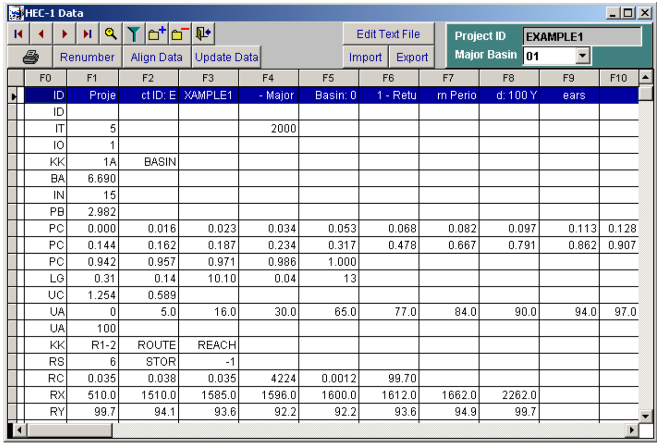
The HEC-1 data is used for all return periods. When running the model, the appropriate rainfall data is inserted for the particular return period.

When '*Update HEC-1 Data'* is selected, the sub basin and routing data is updated from the relevant Tables in the Database. ***Do not use 'Update HEC-1 Data' if you have modified the HEC-1 Data and wish to run the model with this data or do not have supporting sub-basin data in the Project.***

Note: DDMSW uses a special version of HEC-1 to facilitate data management. Using another version of HEC-1 will result in errors during the importing of final results.

Edit HEC-1 Data

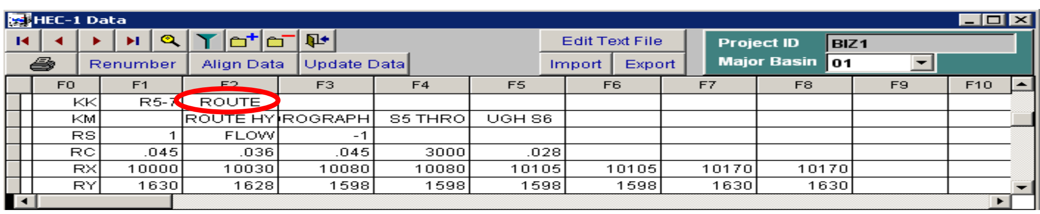
This selection allows the user to add a new HEC-1 file or edit an existing Major Basin HEC-1 file within a project.



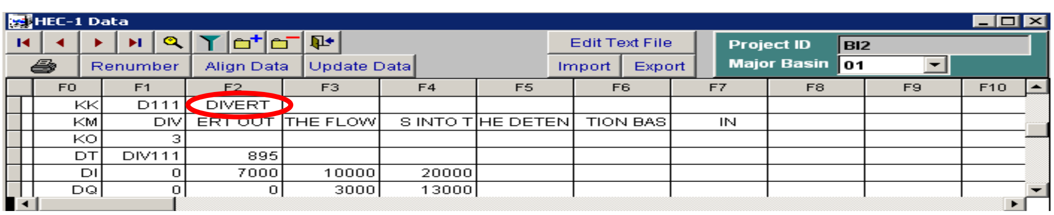
F0	F1	F2	F3	F4	F5	F6	F7	F8	F9	F10
ID	Proje	ct ID: E	XAMPLE1	- Major	Basin: 0	1 - Retu	rn Perio	d: 100 Y	ears	
ID										
IT	5			2000						
IO	1									
KK	1A	BASIN								
BA	6.690									
IN	15									
PB	2.982									
PC	0.000	0.016	0.023	0.034	0.053	0.068	0.082	0.097	0.113	0.128
PC	0.144	0.162	0.187	0.234	0.317	0.478	0.667	0.791	0.862	0.907
PC	0.942	0.957	0.971	0.986	1.000					
LG	0.31	0.14	10.10	0.04	13					
UC	1.254	0.589								
UA	0	5.0	16.0	30.0	65.0	77.0	84.0	90.0	94.0	97.0
UA	100									
KK	R1-2	ROUTE	REACH							
RS	6	STOR	-1							
RC	0.035	0.038	0.035	4224	0.0012	99.70				
RX	510.0	1510.0	1585.0	1596.0	1600.0	1612.0	1662.0	2262.0		
RY	99.7	94.1	93.6	92.2	92.2	93.6	94.9	99.7		

The data in this file can be exported to an ASCII file to facilitate editing by clicking on the *"Export"* button and following the instructions on the screen. After the file is exported to an ASCII file, users can use a Text Editor (Wordpad, HEC's COED or other) to arrange the sequence of the KK blocks. When finished with the editing, the file can be imported by clicking on the *"Import"* button. Importing replaces the existing data for the selected Major Basin. To view a different Major Basin's data, use the Major Basin pull-down menu. ***Do not use 'Update Data' if you have modified the HEC-1 Data and wish to run the model with this data or do not have supporting sub-basin data in the Project.***

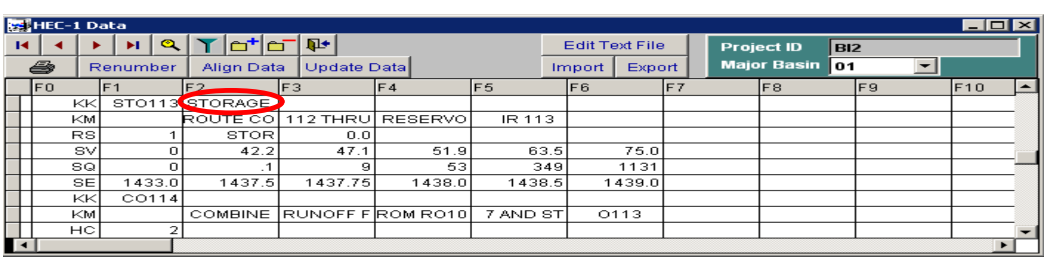
When importing a HEC-1 file where there is no supporting soil, landuse and sub-basin data, it is necessary to add "Route", "Divert" or "Storage" to the F2 field of the KK card in the HEC-1 file to make use of the graphing functions for these functions as shown below.



F0	F1	F2	F3	F4	F5	F6	F7	F8	F9	F10
KK	R5-7	ROUTE								
KM		ROUTE HY	ROGRAPH	S5 THRO	UGH S6					
RS	1	FLOW	-1							
RC	.045	.036	.045	3000	.028					
RX	10000	10030	10080	10080	10105	10105	10170	10170		
RY	1630	1628	1598	1598	1598	1598	1630	1630		



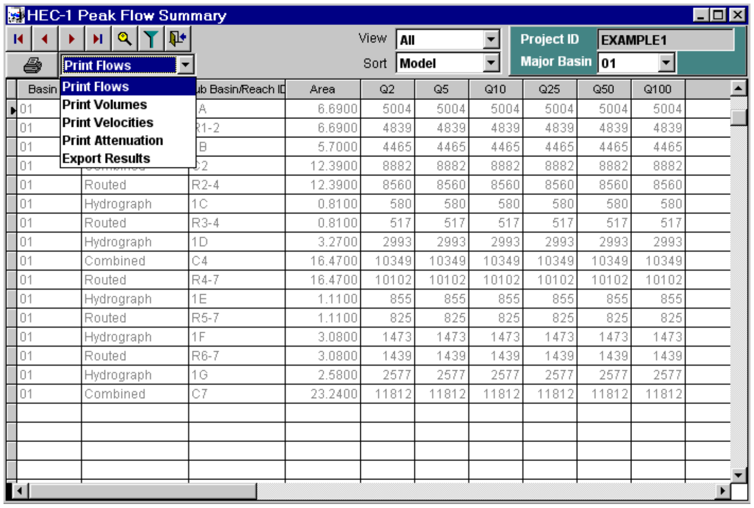
F0	F1	F2	F3	F4	F5	F6	F7	F8	F9	F10
KK	D111	DIVERT								
KM	DIV	ERT OUT	THE FLOW	S INTO T	HE DETEN	TION BAS	IN			
KO	3									
DT	DIV111	895								
DI	0	7000	10000	20000						
DQ	0	0	3000	13000						



F0	F1	F2	F3	F4	F5	F6	F7	F8	F9	F10
KK	STO113	STORAGE								
KM		ROUTE CO	112 THRU	RESERVO	IR 113					
RS	1	STOR	0.0							
SV	0	42.2	47.1	51.9	63.5	75.0				
SQ	0	.1	9	53	349	1131				
SE	1433.0	1437.5	1437.75	1438.0	1438.5	1439.0				
KK	CO114									
KM		COMBINE	RUNOFF F	ROM RO10	7 AND ST	O113				
HC	2									

View Summary Results

This displays summary results of model runs. The data cannot be edited.



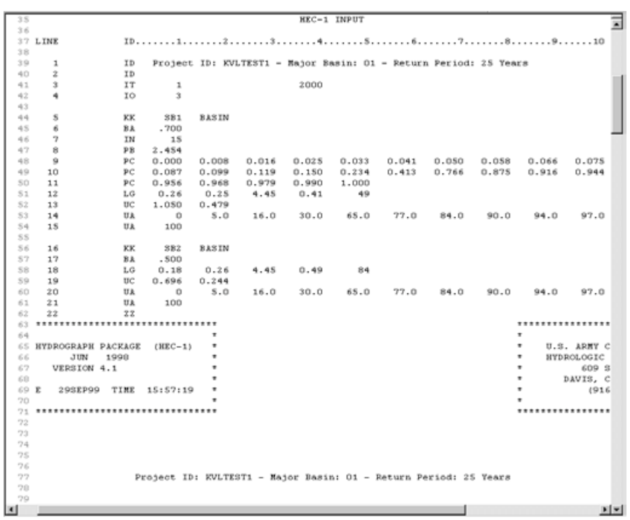
Basin		ub Basin/Reach ID	Area	Q2	Q5	Q10	Q25	Q50	Q100
01		A	6.6900	5004	5004	5004	5004	5004	5004
01		R1-2	6.6900	4839	4839	4839	4839	4839	4839
01		B	5.7000	4465	4465	4465	4465	4465	4465
01		C2	12.3900	8882	8882	8882	8882	8882	8882
01	Routed	R2-4	12.3900	8560	8560	8560	8560	8560	8560
01	Hydrograph	1C	0.8100	580	580	580	580	580	580
01	Routed	R3-4	0.8100	517	517	517	517	517	517
01	Hydrograph	1D	3.2700	2993	2993	2993	2993	2993	2993
01	Combined	C4	16.4700	10349	10349	10349	10349	10349	10349
01	Routed	R4-7	16.4700	10102	10102	10102	10102	10102	10102
01	Hydrograph	1E	1.1100	855	855	855	855	855	855
01	Routed	R5-7	1.1100	825	825	825	825	825	825
01	Hydrograph	1F	3.0800	1473	1473	1473	1473	1473	1473
01	Routed	R6-7	3.0800	1439	1439	1439	1439	1439	1439
01	Hydrograph	1G	2.5800	2577	2577	2577	2577	2577	2577
01	Combined	C7	23.2400	11812	11812	11812	11812	11812	11812

Clicking on *"All, Combined Hydrograph or Routed"*, will filter the data to the selection. The data can also be ordered by *"Model or Numeric"*, where 'Model' is the natural order from the model results and 'Numeric' is sorted by Sub Basin or Reach ID.

The user can view results by selecting 'Print Flows', 'Print Volumes', 'Print Velocities' or 'Print Attenuation' and clicking the print button. To print the report to a printer, click on the printer icon. To export the report to a PDF or other file format, click on the export button next the the printer setup icon. The default filename is the Project ID plus the Major Basin separated by "-". The default location is the 'temp' sub directory for the project. The user can override these by entering a specific filename and location for the database file.

View Output File

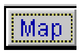
Select a file from the file selection dialogue box. The text editor opens the ASCII model output file. When finished viewing, close the window by clicking ☒, otherwise the text editor program will remain loaded in memory.



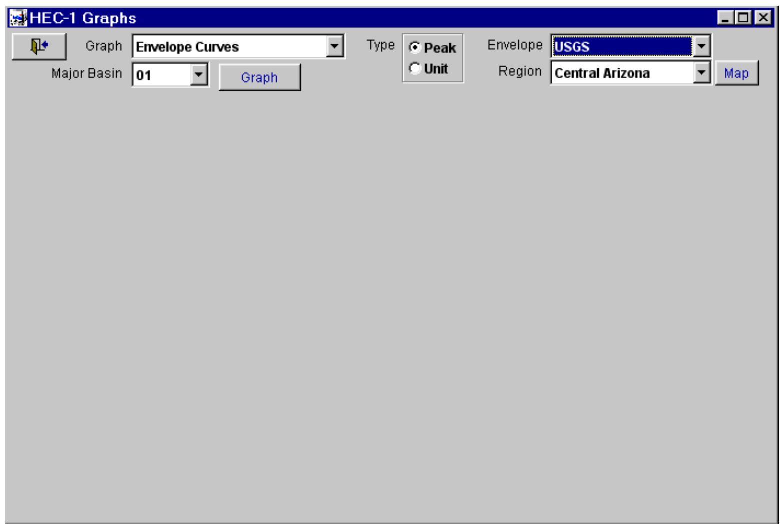
```
                                          HEC-1 INPUT

 LINE          ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10

   1           ID   Project ID: KVLTEST1 - Major Basin: 01 - Return Period: 25 Years
   2           ID
   3           IT       1                   2000
   4           IO       3

   5           KK     SB1   BASIN
   6           BA    .700
   7           IN      15
   8           PB   2.454
   9           PC   0.000   0.008   0.016   0.025   0.033   0.041   0.050   0.058   0.066   0.075
  10           PC   0.087   0.099   0.119   0.150   0.234   0.413   0.766   0.875   0.916   0.944
  11           PC   0.956   0.968   0.979   0.990   1.000
  12           LG    0.26    0.25    4.45    0.41      49
  13           UC   1.050   0.479
  14           UA       0     5.0    16.0    30.0    65.0    77.0    84.0    90.0    94.0    97.0
  15           UA     100

  16           KK     SB2   BASIN
  17           BA    .500
  18           LG    0.18    0.26    4.45    0.49      84
  19           UC   0.696   0.244
  20           UA       0     5.0    16.0    30.0    65.0    77.0    84.0    90.0    94.0    97.0
  21           UA     100
  22           ZZ
*******************************                                                      ****************
                              *                                                      *
HYDROGRAPH PACKAGE  (HEC-1)   *                                                      *    U.S. ARMY C
     JUN   1998               *                                                      *    HYDROLOGIC
   VERSION 4.1                *                                                      *         609 S
                              *                                                      *       DAVIS, C
E   29SEP99  TIME  15:57:19   *                                                      *          (916
                              *                                                      *
*******************************                                                      ****************





                 Project ID: KVLTEST1 - Major Basin: 01 - Return Period: 25 Years


```

Graphs

The 'Graphs' option has been developed to facilitate the review of hydrological results and data input. Graphs that facilitate the review of hydrological results include envelope curves for USGS, Malvick and Boughton. These curves are compared to peak or unit discharge model results. Graphs that facilitate the review of input data include channel cross-sections, diversions, stage-discharge and stage-volume relationships.

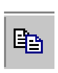
Click 

to view the map of the USGS region.

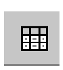


Graphs Toolbar

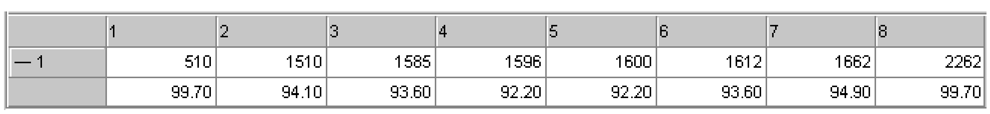
The following tools are available on the graph screen:



Copies the graph to the clipboard as a bitmap, metafile, text or OLE object.



Data Editor. This displays the data values at the bottom of the screen. These values can be edited, and the graph dynamically reflects these changes.

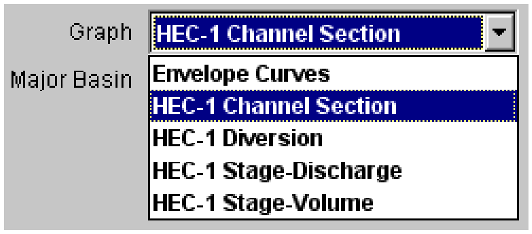


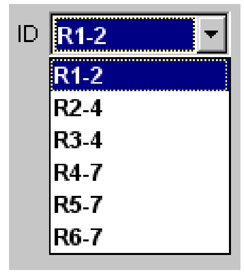
	1	2	3	4	5	6	7	8
— 1	510	1510	1585	1596	1600	1612	1662	2262
	99.70	94.10	93.60	92.20	92.20	93.60	94.90	99.70

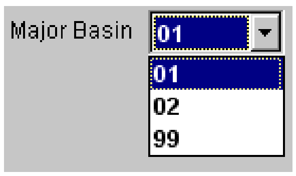
Zoom tool. Click this icon and draw an area of the graph to be magnified.

Prints the graph.

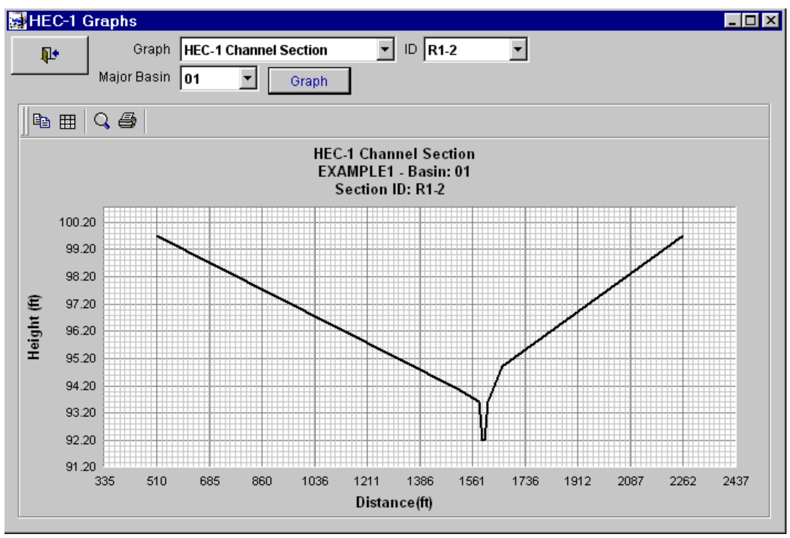
Select a Major Basin, the Section ID and the type of graph from the pull-down lists.





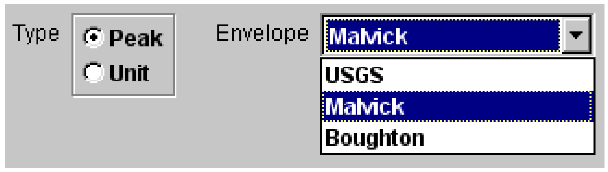


Click the  button to display the graph on the screen.



Depending on the graph type, other choices are available as follows:

Envelope Curves (USGS, Malvick and Boughton)

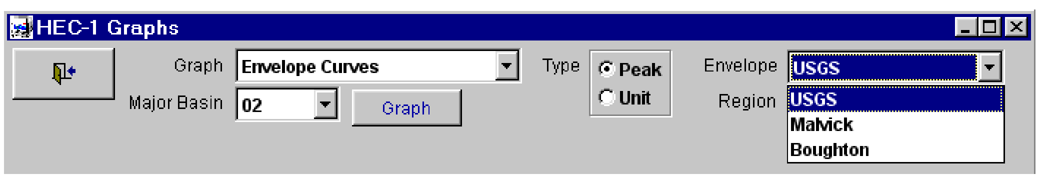


Select USGS, Malvick or Boughton data. The USGS data was derived manually from the data contained in “Methods for Estimating Magnitude and Frequency of Floods in the Southwestern United States”, United States Geological Survey Water-Supply Paper 2433. Northeastern Arizona – Figure 39, Central Arizona – Figure 41, Southern Arizona – Figure 42 and Upper Gila Ben – Figure 44. The legend on each of the graphs is as follows:

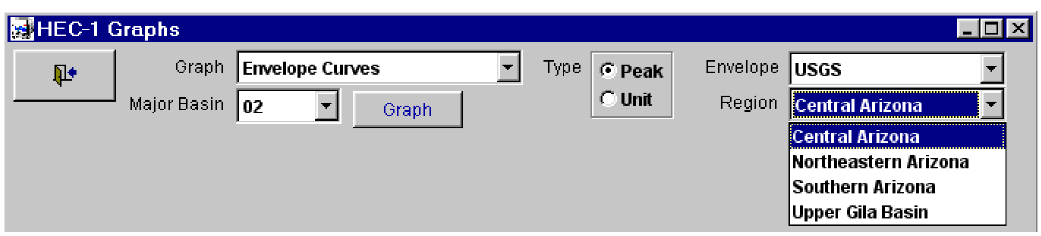
Envelope	Envelope Curve for Study Area
Region	100-Year Peak Discharge Relation for Region
Low-Mid Elevation	100-Year peak Discharge Relation for Low to Middle-Elevation Study Area

The Malvick data was derived from “A Magnitude-Frequency-Area relation for Floods in Arizona”, Allan J. Malvick, January 1980, Figure 6 – 100-year curve. The Boughton data was derived from “Highway Drainage Design Manual Hydrology”, Report Number FHWA-AZ93-281, March 1993, Figure 10-1 Curve H.

The selection of Envelope Curves from the Graphs pull-down provides the following choices.



If USGS is selected from the Envelope pull-down, then the user can select the appropriate Region as shown.

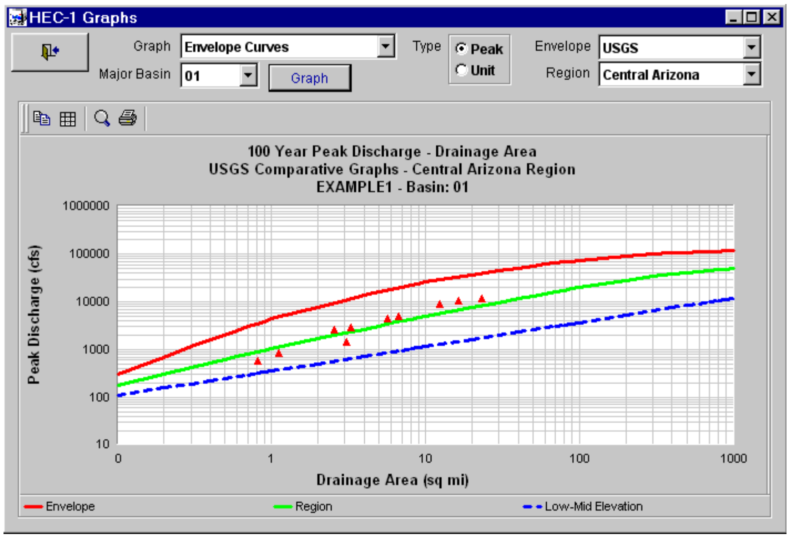


All of the envelope curves can be graphed in either Peak (Peak Discharge in cfs) or Unit (Unit Discharge in cfs/sq mi).

Peak Discharges Scatter Graph

The Peak Discharges Graph is a graph of drainage area verses peak discharge (or unit discharges) for sub-basins and combined flows. The data is generated when running HEC-1 and is saved in Table HEC1SUMM.DBF. For this analysis, only the 100-year flows are graphed. From within a specific Project, the user selects a major basin. If the major basin has been modeled, the 100-year area and discharges for "*Hydrographs"* (sub-basins) and "*Routed"* (routed flows) are copied to a temporary file for graphing.

Selection of Major Basin 01, USGS Envelope with Central Region and Peak will produce the following graph:



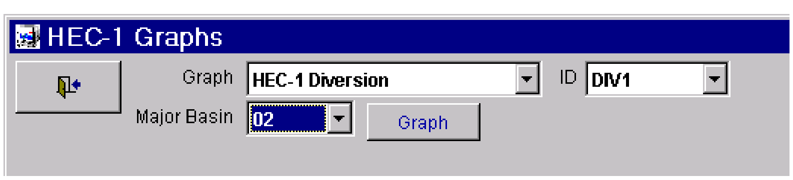
Note that the legend can moved anywhere on the graph screen.

Rating Curves Plot

Rating curves are for Stage-Discharge, Stage-Volume and Diversions. The data is developed from the HEC-1 input file as follows:

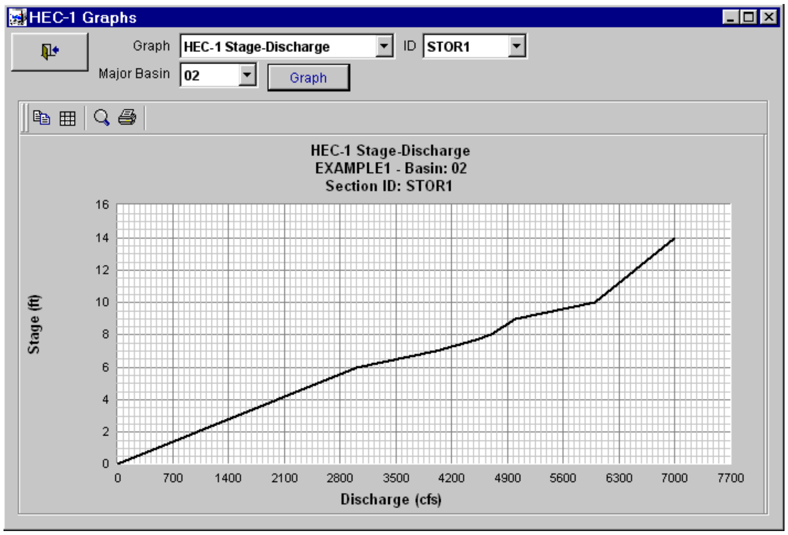
Stage-Discharge	SE and SQ cards
Stage-Volume	SE and SV cards
Diversions	DI and DQ cards

When the user selects a new Major Basin, the HEC-1 input file for the Major Basin is scanned for SE and DI cards. The KK value for all found SE and DI cards establishes the available ID's to graph. The ID's are saved to a temporary file and the data for the graphs (SE-SQ, SE-SV and DI-DQ) are saved to a separate temporary file. The user must also select an ID.

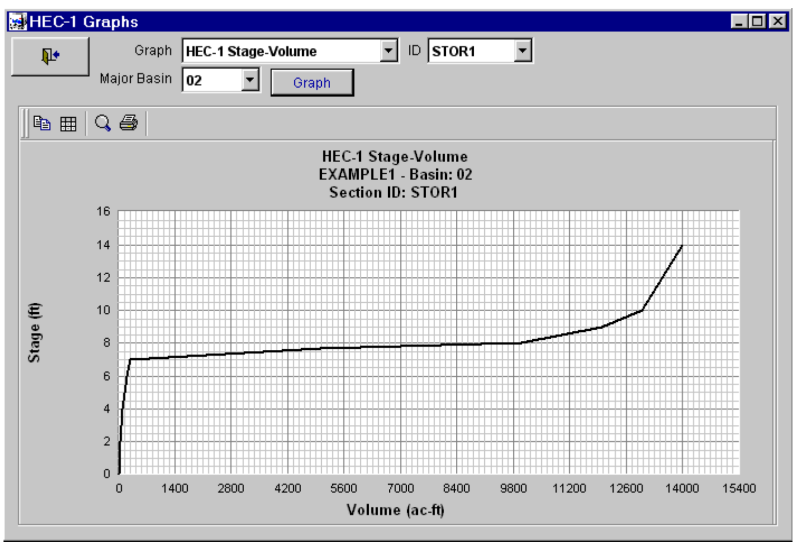


The following three Rating Curve graphs are available:

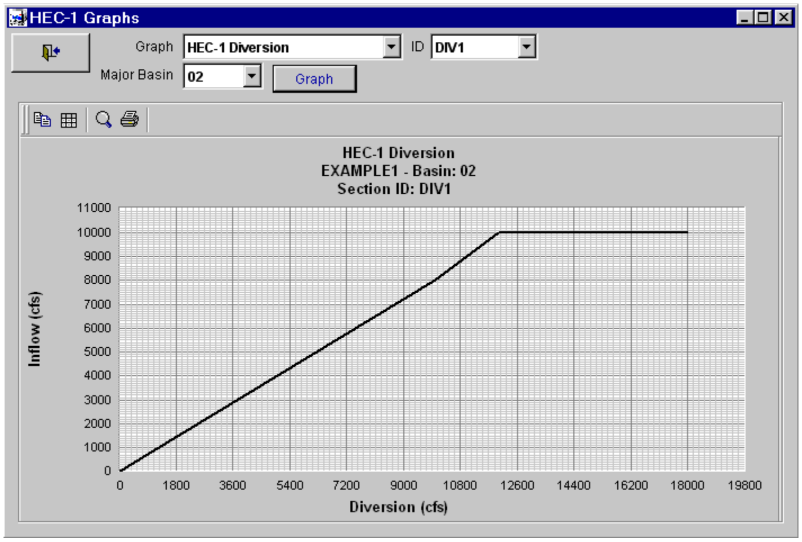
Stage-Discharge



Stage-Volume

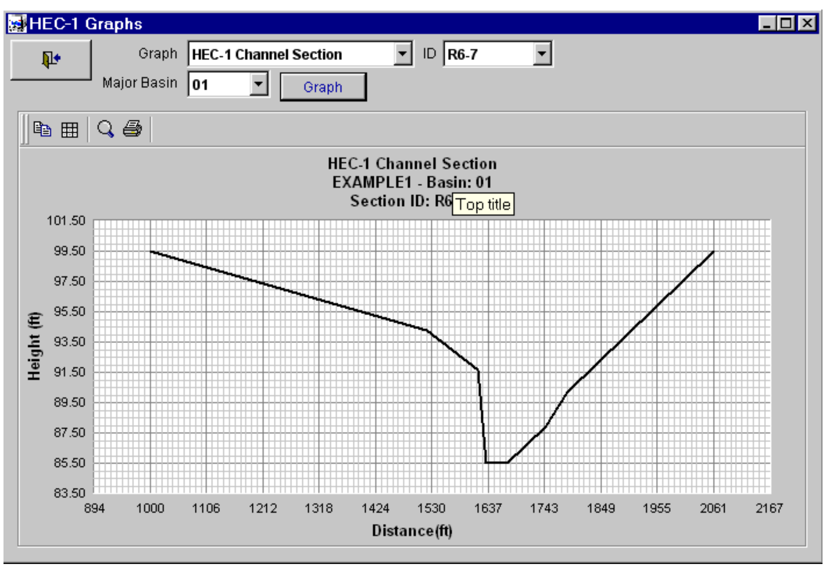


Diversion



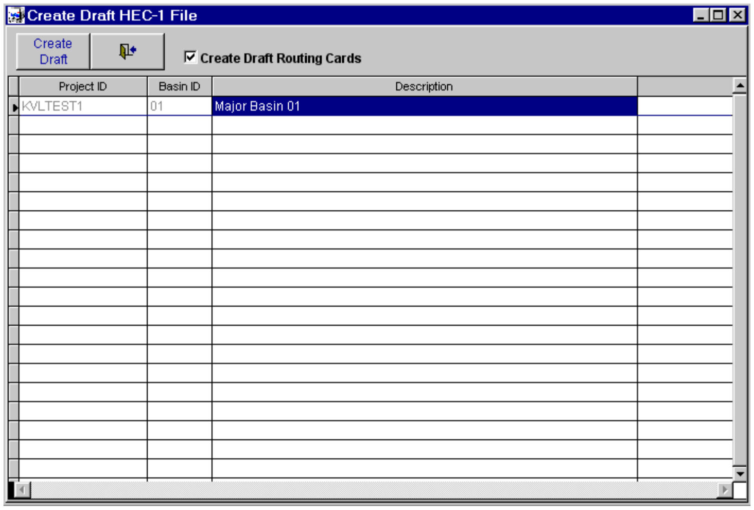
Channel Section Plot

Channel section plots are for RX-RY cards developed from the HEC-1 input file. When the user selects a new Major Basin, the HEC-1 input file for the Major Basin is scanned for RX cards. The KK value for all found RX cards establishes the available ID's to graph. The ID's are saved to a temporary file and the data for the graphs (RX-RY) are saved to a separate temporary file. The user selects an ID for the following typical graph.



Develop Draft Model Data

The selection of Develop Draft Model Data from the Hydrology\HEC-1 menu is used to develop draft HEC-1 data from the Sub Basin and default data used in the project. When "Create Draft" button is clicked, the program **will replace any existing data for the selected Major Basin** in the HEC-1 data file and should only be used at the beginning of a project. The program requires the Sub Basin data to be available. If *"Create Draft Routing Cards"* is selected, available routing data will be appended to the draft file. It will be up to the user to place the routing cards in the appropriate location.



5. Hydraulics

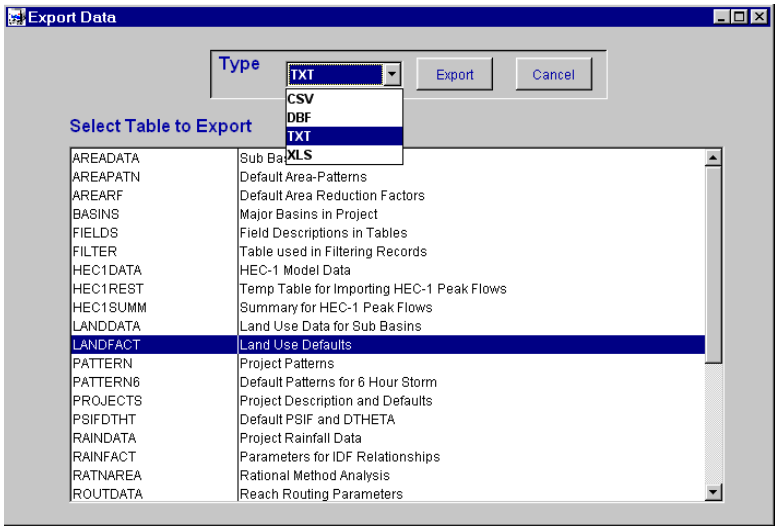
To be developed!

6. GIS

To be developed!

7. Utilities

Export Data



The selection of Export Data from the Utilities menu is used to export a Table from the Database to a file in a different format. All the record data in the Table are exported. Note that the Table contents are not removed, but are copied to a different file format. The file formats supported include:

CSV Data is exported into a file one record per line, with each field separated by a comma. Character fields are enclosed in quotes. For example:

```
projectid,lucode,dthetadesc,vegcover,rtimp,ia,kn,kbdesc,group,descript,c10,iadesc,sort
"BETA010","M","Dry",25.0,0,0.25,0.050,"Max","","MOUNTAIN",0.00,"",10
"BETA010","H","Dry",30.0,0,0.15,0.040,"Hi","","HILLSLOPE",0.00,"",20
"BETA010","D","Dry",30.0,0,0.35,0.030,"Low","","DESERT RANGELAND",0.00,"",30
```

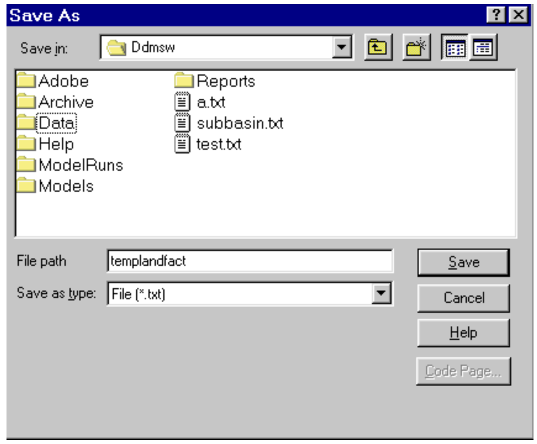
TXT Data is exported one record per line in ASCII text that can be read by any text editor. The data is in fixed format (columns). For example:

```
BETA010         M               Dry     25.0   0 0.250.050Max                               MOUNTAIN
BETA010         H               Dry     30.0   0 0.150.040Hi                                HILLSLOPE
BETA010         D               Dry     30.0   0 0.350.030Low                               DESERT RANGELAND
```

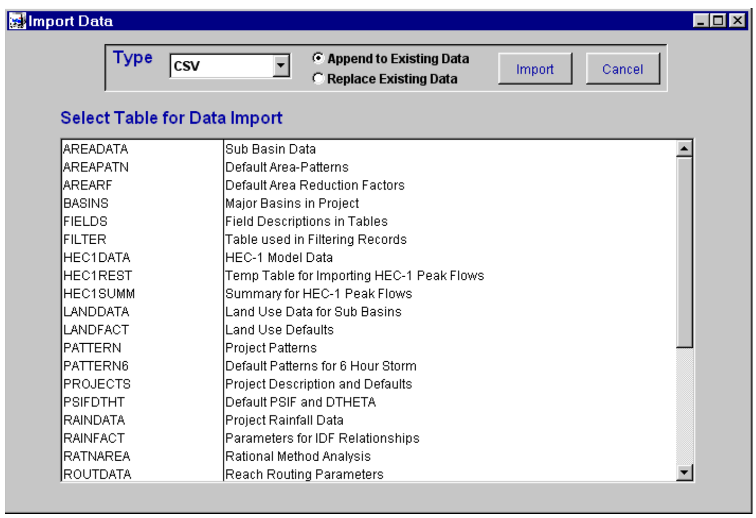
DBF Exports data into a format that can be read by a Database program such as Dbase or FoxPro.

XLS Select this type to create an spreadsheet which can be opened in Microsoft Excel.

Use the mouse to highlight the Table and select an export type from the drop down selection box, then click ‘Export’. The contents (all records) of the selected Table is saved on disk in the chosen format. The user is prompted to enter a filename and location for the export file.



Import Data

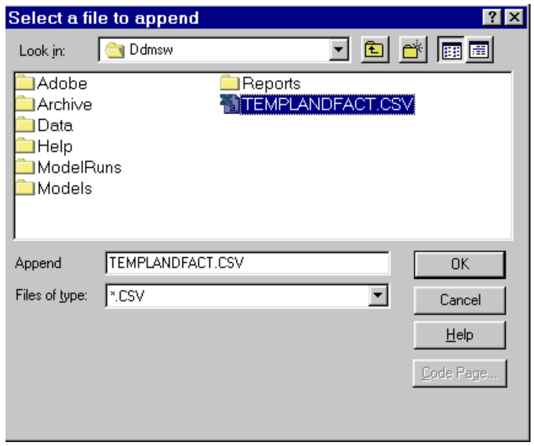


The selection of Import Data from the Utilities menu is used to import data originating from another source, such as a spreadsheet, text file or another database, into a selected table. The file formats supported include CSV, TXT, DBF, and XLS, as described in ‘Export Data’. There are two options:

- Append to Existing Data — This option <u>adds</u> the data to the existing data. The results will include the current and the newly imported data.
- Replace Existing Data — This option <u>deletes</u> the current data in the table, and then <u>replaces</u> it with the new data.

It is important that the data to be imported has the <u>same structure</u> as the importing Table, otherwise fields will be truncated and records rejected. For this reason, a good practice is to first export a Table and then use the exported file as a template for the acceptable format.

Highlight the Table to append or replace, select the file type and click 'Import'. The following dialogue box appears for the user to select the file to import. Highlight the filename and click 'OK'.



Pack Tables

This function rebuilds the Table indexes, and then packs all Tables in the Database. Packing is the process of **permanently removing records** that have been marked for deletion. Once a Table is packed, records cannot be recovered. Packing recoups disk space occupied by deleted records. It should not be necessary to pack the Database frequently, but on occasion after many records have been deleted over time.

> **!** **Caution:** **Packing Tables take a few minutes to complete. Do not interrupt the process once it has begun.**

Table Descriptions

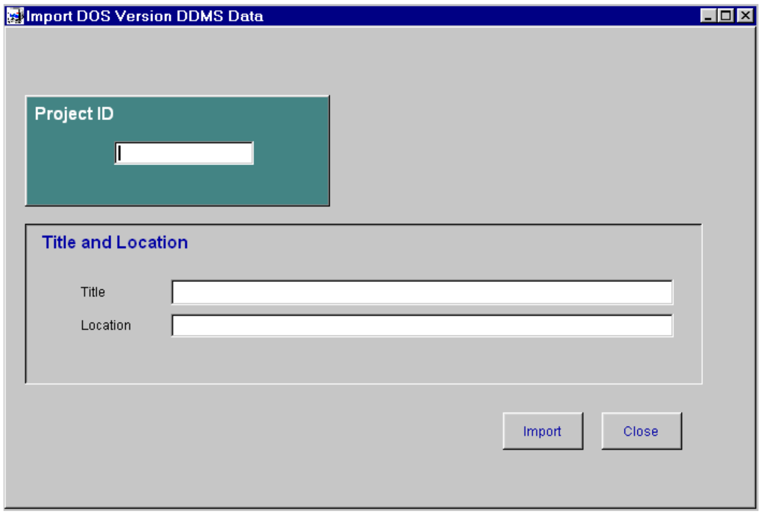
The selection of Table Descriptions from the Utilities menu is used to view the name and description of Tables used in the application. The data cannot be edited.

Field Descriptions

The selection of Field Descriptions from the Utilities menu is used to view the structure of each Table used in the application. The data cannot be edited.

Import DOS Family

The selection of Import DOS Family from the Utilities menu is used to import data developed in the DOS version of DDMS.



Enter a Project ID, title and location for the data to be imported. This is the same information as starting a new project. The click the *"Import"* button. The program will import data as follows:

Sub Basin Area data	From a file with an .SBR extension
Default Land Use data	From a file with an .LDF extension
Sub Basin Land Use data	From a file with an .SUB extension
Sub Basin Soil data	From a file with an .SUB extension
Hydrograph Type	From a file with an .SUB extension
Precipitation data	From a file with an .PFI extension
Storms, duration,Timearea,NMIN	From either *M1I or *M2I

If the above files are not available for the *"family",* then not all of the required data will be imported and the remaining required data will have to be entered manually into the Database.

8. Help and Window

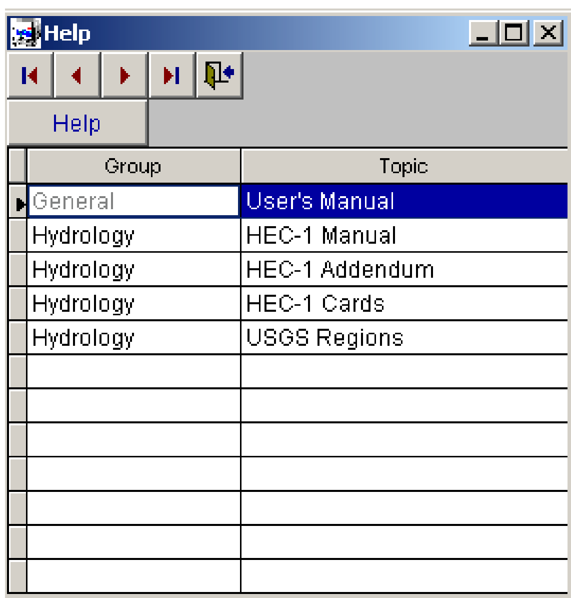
Help

About

This displays the informational screen for the application.

Help

This option on the Help menu displays a list of manuals that the user can view. Highlight a manual and click Help. Acrobat reader opens the manual on the screen. When finished viewing, close the Acrobat screen otherwise it will remain open on the desktop. (The path to Acrobat Reader must be entered in the Help Reader in File\Setup.)



Window

Two options are available on the Window Menu.

Cascade

Use this to cascade all open forms on the screen. Alternatively, the user can press 'Ctrl T' at any time to arrange forms.

Close All

Use this to close all open forms. Pressing 'Ctrl A' has the same effect.

Example

Introduction

There are a number of Projects that have been used to test the application. The data in these projects can be reviewed by importing the Project to see how data is entered for the various type of default settings. The following are Projects that come with DDMSW installation:

Project ID	Hydrograph	Storms	Duration	Loss Method
EXAMPLE1	Clark	Single	6 Hour	Green-Ampt
EXAMPLE2	S-Graph	Single	24 Hour	Green-Ampt
EXAMPLE3	S-Graph	Multiple	6 Hour	Green-Ampt
EXAMPLE4	Clark	Single	6 Hour	Init & Uniform
EXAMPLE5	Rational Method			

Establish a New Project (MYEXAMPLE1)

The steps to establish a new project through to running the HEC-1 model and viewing the results are as follows:

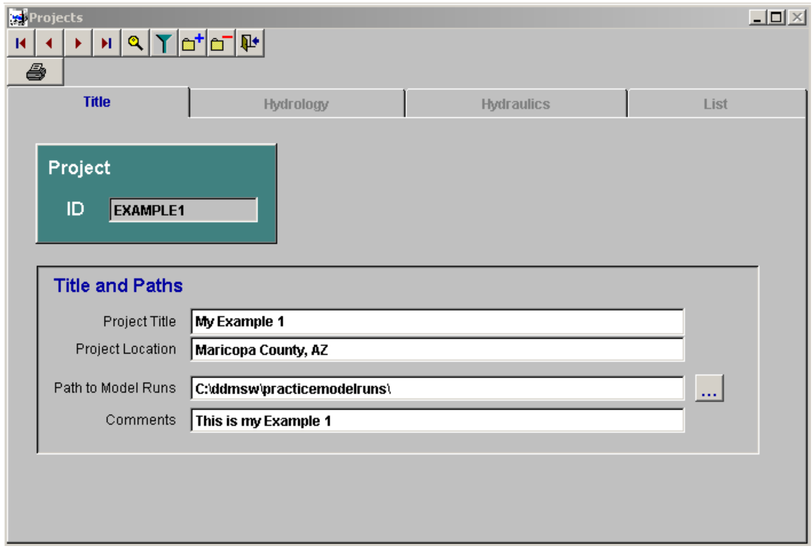
1. Establish Path for Model Runs
2. Create New Project and Establish Defaults
3. Establish Rainfall Data for Project
4. Establish Land Use Defaults
5. Establish Soil Defaults
6. Establish Land Use Data
7. Establish Soil Data
8. Establish Major Basin
9. Establish Sub Basin Data
10. Establish Routing Data
11. Develop Draft HEC-1 Input File
12. Edit Draft HEC-1 Input File
13. Run HEC-1 Model
14. View Model Summary Results

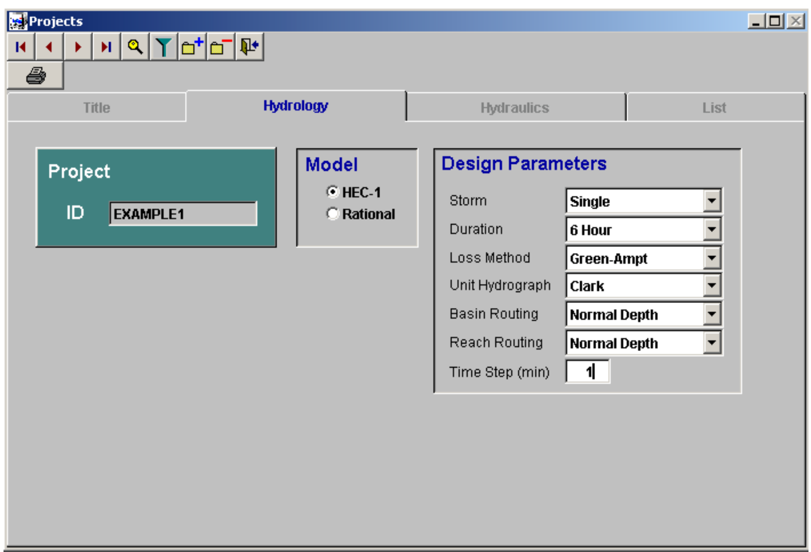
1. Establish Path for Model Runs

Create a folder for the HEC-1 model runs. For this example, a new folder C:\ddmsw\practicemodelruns has been created.

2. Create New Project and Establish Defaults

Select *New Project* from the *File* Menu and fill in data for Title and Hydrology defaults.

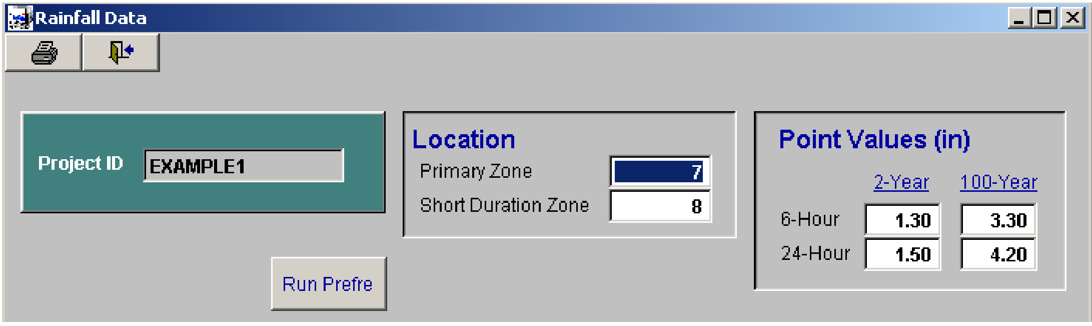




In Maricopa County, Basin Routing is seldom used but may be occasionally used for overland flow.

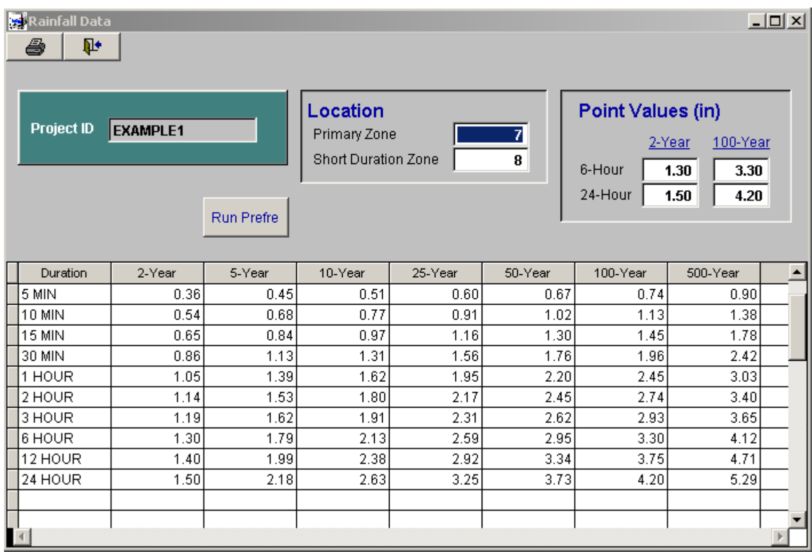
3. Establish Rainfall Data for Project

Select *Prefre* from the *Hydrology\Rainfall* Menu. Fill in data shown below.



Click 

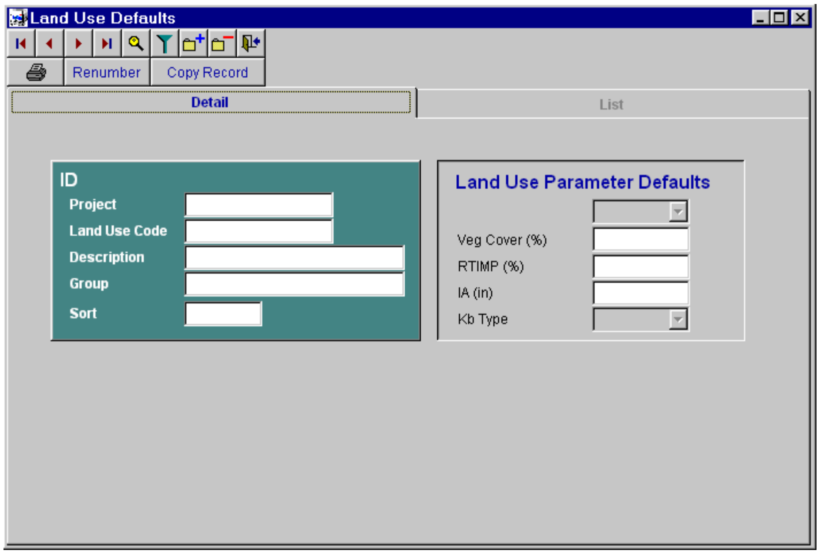
to run the Prefre model and establish rainfall data as shown below.



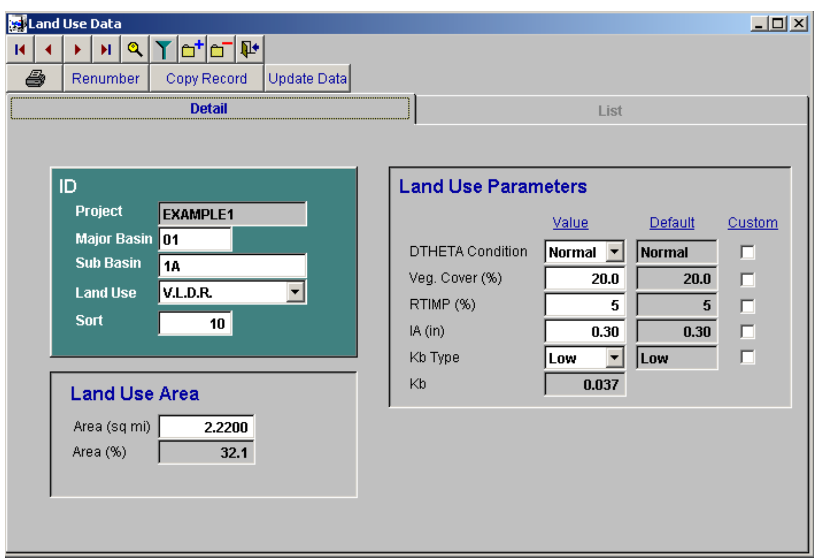
Duration	2-Year	5-Year	10-Year	25-Year	50-Year	100-Year	500-Year
5 MIN	0.36	0.45	0.51	0.60	0.67	0.74	0.90
10 MIN	0.54	0.68	0.77	0.91	1.02	1.13	1.38
15 MIN	0.65	0.84	0.97	1.16	1.30	1.45	1.78
30 MIN	0.86	1.13	1.31	1.56	1.76	1.96	2.42
1 HOUR	1.05	1.39	1.62	1.95	2.20	2.45	3.03
2 HOUR	1.14	1.53	1.80	2.17	2.45	2.74	3.40
3 HOUR	1.19	1.62	1.91	2.31	2.62	2.93	3.65
6 HOUR	1.30	1.79	2.13	2.59	2.95	3.30	4.12
12 HOUR	1.40	1.99	2.38	2.92	3.34	3.75	4.71
24 HOUR	1.50	2.18	2.63	3.25	3.73	4.20	5.29

4. Establish Land Use Defaults

Select *Defaults* from the *Hydrology\Land Use* Menu. The first time you come to this screen, it will look like the following screen. This screen will look different if "Green-Ampt" is not the default Loss Method.



Click  to create a new record and fill in the data for the first record as shown below. Use appropriate Tables in the County's Drainage Manual for reference.

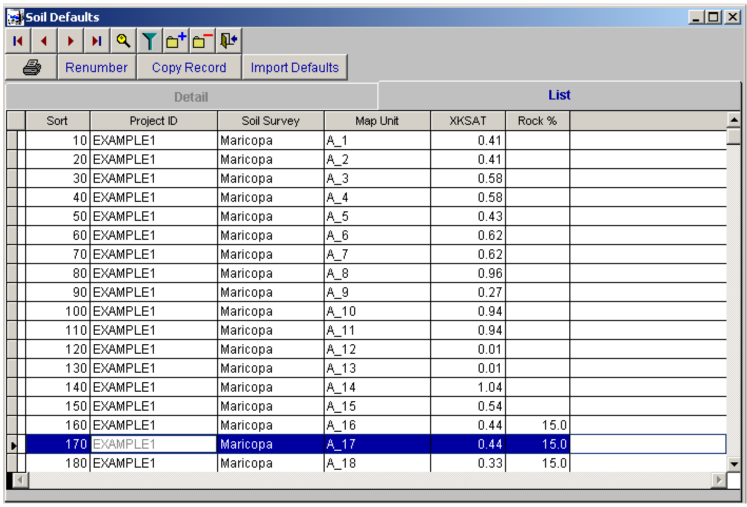


To add new records either create a new record as just described or click 

(use Browse to view data) and edit the data for the new record.

5. Establish Soil Defaults

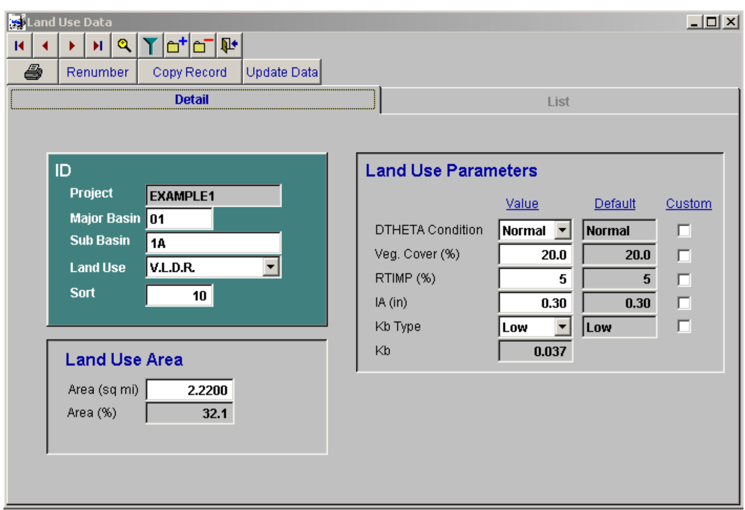
Select *Defaults* from the *Hydrology\Soil* Menu. If Soil Default data does not exist for this project, then the County default Table will be loaded. The user can then modify the data to establish different defaults to be used for this project. If it is necessary to add new data, then do it in the same manner as adding new records for the Land Use Defaults. The following view is shown in Browse mode.



Sort	Project ID	Soil Survey	Map Unit	XKSAT	Rock %
10	EXAMPLE1	Maricopa	A_1	0.41	
20	EXAMPLE1	Maricopa	A_2	0.41	
30	EXAMPLE1	Maricopa	A_3	0.58	
40	EXAMPLE1	Maricopa	A_4	0.58	
50	EXAMPLE1	Maricopa	A_5	0.43	
60	EXAMPLE1	Maricopa	A_6	0.62	
70	EXAMPLE1	Maricopa	A_7	0.62	
80	EXAMPLE1	Maricopa	A_8	0.96	
90	EXAMPLE1	Maricopa	A_9	0.27	
100	EXAMPLE1	Maricopa	A_10	0.94	
110	EXAMPLE1	Maricopa	A_11	0.94	
120	EXAMPLE1	Maricopa	A_12	0.01	
130	EXAMPLE1	Maricopa	A_13	0.01	
140	EXAMPLE1	Maricopa	A_14	1.04	
150	EXAMPLE1	Maricopa	A_15	0.54	
160	EXAMPLE1	Maricopa	A_16	0.44	15.0
170	EXAMPLE1	Maricopa	A_17	0.44	15.0
180	EXAMPLE1	Maricopa	A_18	0.33	15.0

6. Establish Land Use Data

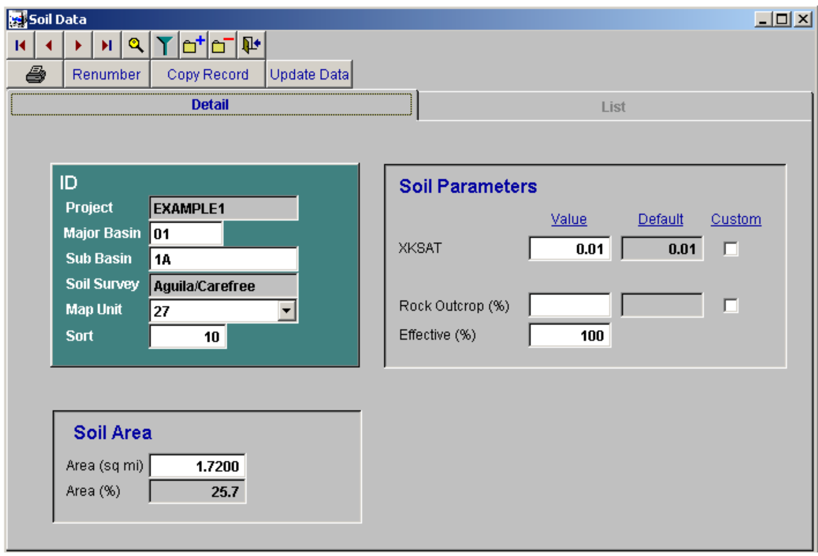
Select *Data* from the *Hydrology\Land Use* Menu. Add or Copy records to populate the necessary data as shown below. It is only necessary to add the Sub Basin ID, select the Land Use and add the Area in square miles. Then click Update Data to populate the Default Values. If a non-default value is used, it will be necessary to check the adjacent Custom box. Values with adjacent Custom box checked will not be updated.



Be sure to enter land use data for each Sub Basin ID in this Project and make sure there is sufficient land use data to cover the entire Sub Basin.

7. Establish Soil Data

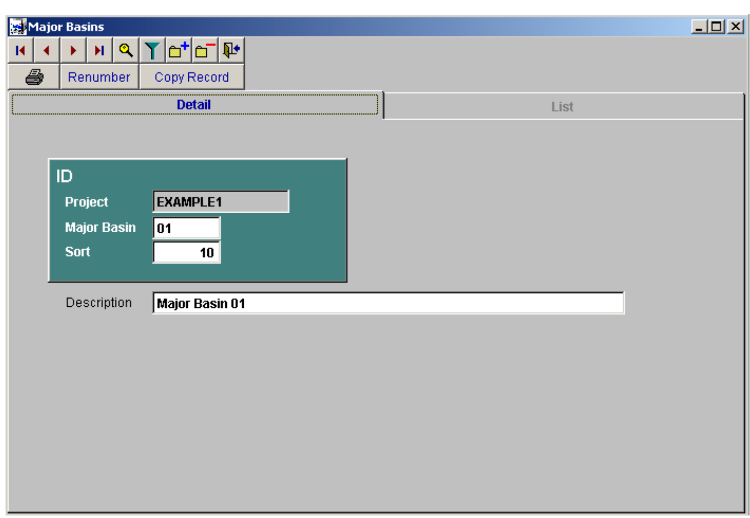
Select *Data* from the *Hydrology\Soil* Menu. Add or Copy records to populate the necessary data as shown below. It is only necessary to add the Sub Basin ID, select the Map Unit and add the Area in square miles. Then click Update Data to populate the Default Values. If a non-default value is used, it will be necessary to check the adjacent Custom box. Values with adjacent Custom box checked will not be updated.



Be sure to enter soil data for each Area ID in this Project and make sure there is sufficient soil data to cover the entire Area ID's area.

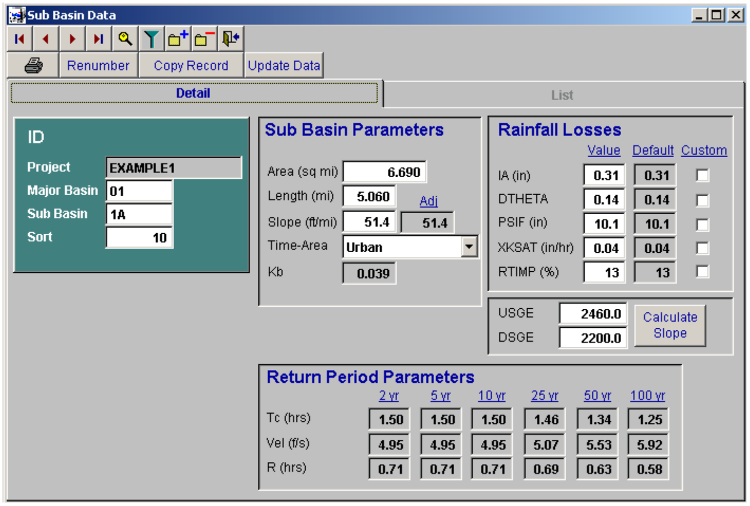
8. Establish Major Basin

Select Major Basins from the *Hydrology\Basins* Menu. Add or Copy records to populate the necessary data as shown below. Each hydrology model run will be for a unique Major Basin ID. Select 01 for the first basin, 02 through 99 for other Major Basins.



9. Establish Sub Basin Data

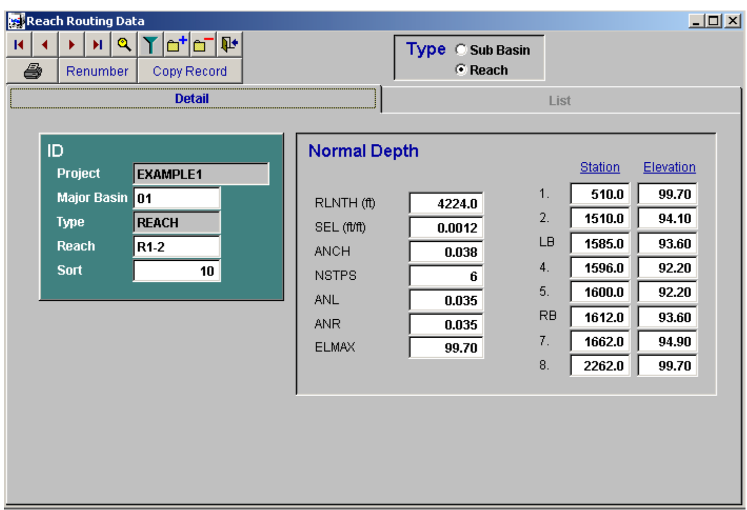
Select *Sub Basins* from the *Hydrology\Basins* Menu. Add or Copy records to populate the necessary data as shown below. Forms may look different depending on the established project defaults. To populate the remaining data, click “Update Data”. This updates all records for the Sub Basin data for this Project.



Following the Update Data, if there are any errors or values falling outside standards, then a report will come to the screen that can be printed. Review this report to see what needs to be fixed.

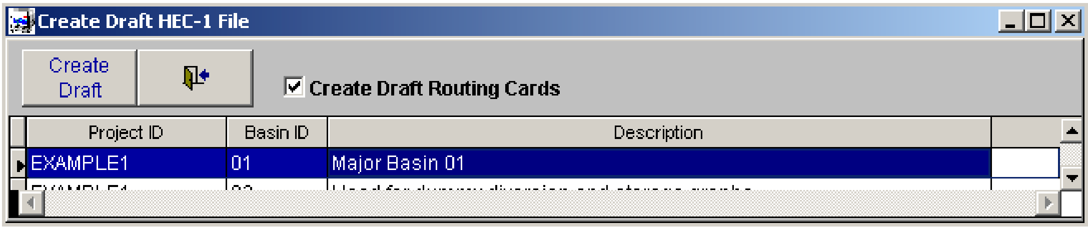
10. Establish Routing Data

Select *Routing* from the *Hydrology\Basins* Menu. Add or Copy records to populate the necessary data as shown below. Forms may look different depending on the established project defaults.



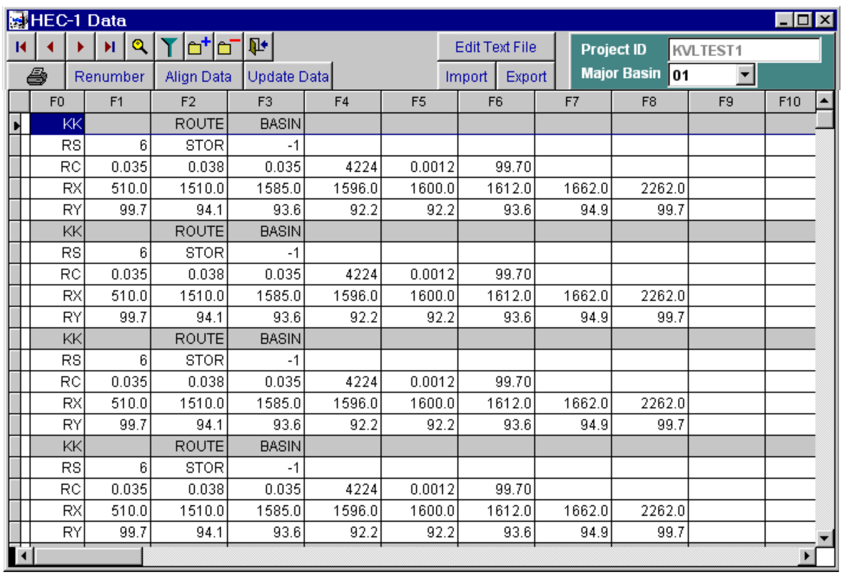
11. Develop Draft HEC-1 Input File

Select *Develop Draft Model Data* from the *Hydrology\HEC-1* Menu. Select the appropriate Major Basin and check whether or not to create Draft Routing Cards. Click on "Create Draft**".**



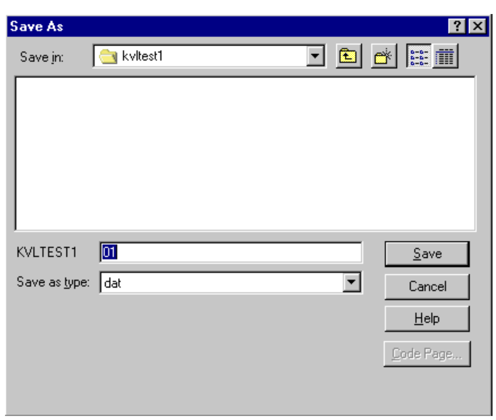
12. Edit Draft HEC-1 Input File

Select *Edit HEC-1 Data* from the *Hydrology\HEC-1* Menu. Select the appropriate Major Basin ID.

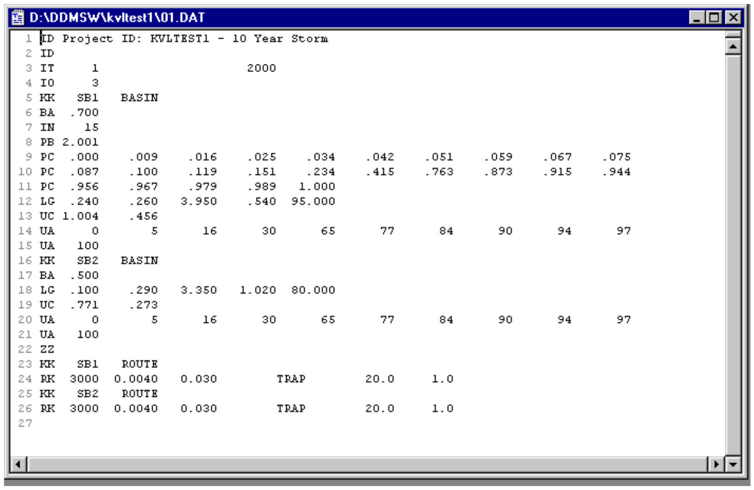


F0	F1	F2	F3	F4	F5	F6	F7	F8	F9	F10
KK		ROUTE	BASIN							
RS	6	STOR	-1							
RC	0.035	0.038	0.035	4224	0.0012	99.70				
RX	510.0	1510.0	1585.0	1596.0	1600.0	1612.0	1662.0	2262.0		
RY	99.7	94.1	93.6	92.2	92.2	93.6	94.9	99.7		
KK		ROUTE	BASIN							
RS	6	STOR	-1							
RC	0.035	0.038	0.035	4224	0.0012	99.70				
RX	510.0	1510.0	1585.0	1596.0	1600.0	1612.0	1662.0	2262.0		
RY	99.7	94.1	93.6	92.2	92.2	93.6	94.9	99.7		
KK		ROUTE	BASIN							
RS	6	STOR	-1							
RC	0.035	0.038	0.035	4224	0.0012	99.70				
RX	510.0	1510.0	1585.0	1596.0	1600.0	1612.0	1662.0	2262.0		
RY	99.7	94.1	93.6	92.2	92.2	93.6	94.9	99.7		
KK		ROUTE	BASIN							
RS	6	STOR	-1							
RC	0.035	0.038	0.035	4224	0.0012	99.70				
RX	510.0	1510.0	1585.0	1596.0	1600.0	1612.0	1662.0	2262.0		
RY	99.7	94.1	93.6	92.2	92.2	93.6	94.9	99.7		

The best way to edit this data is to export the file to an ASCII file and edit the data and then import the ASCII file when edits are complete. Click **"Export"** to export the file. A dialogue box comes up with the default model runs path and the Basin ID. Click save.

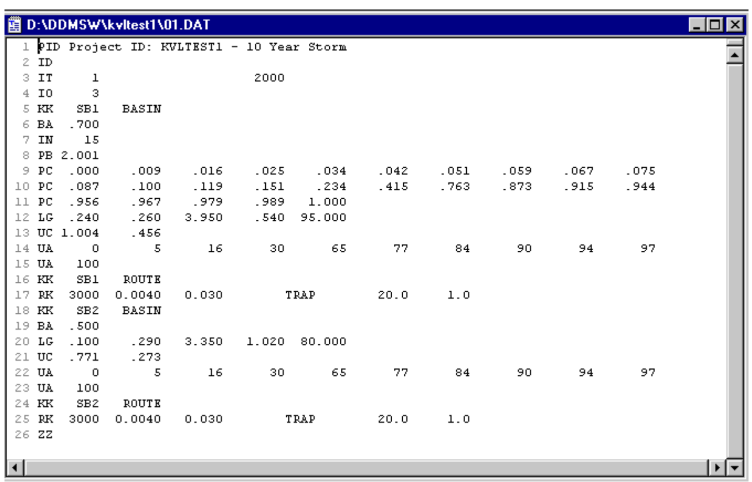


The following is an example of the draft HEC-1 ASCII file (note the routing cards are at the bottom).



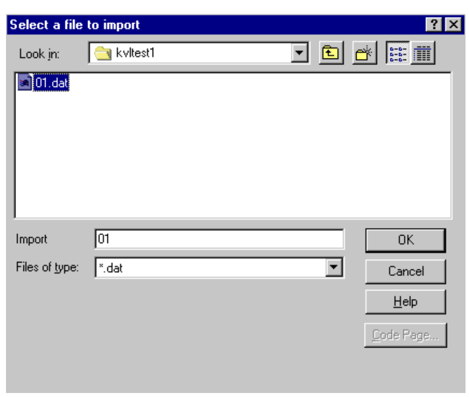

```
ID Project ID: KVLTEST1 - 10 Year Storm
ID
IT     1                  2000
IO     3
KK   SB1   BASIN
BA  .700
IN    15
PB 2.001
PC  .000    .009    .016    .025    .034    .042    .051    .059    .067    .075
PC  .087    .100    .119    .151    .234    .415    .763    .873    .915    .944
PC  .956    .967    .979    .989   1.000
LG  .240    .260   3.950    .540  95.000
UC 1.004    .456
UA     0       5      16      30      65      77      84      90      94      97
UA   100
KK   SB2   BASIN
BA  .500
LG  .100    .290   3.350   1.020  80.000
UC  .771    .273
UA     0       5      16      30      65      77      84      90      94      97
UA   100
ZZ
KK   SB1   ROUTE
RK  3000  0.0040   0.030           TRAP         20.0     1.0
KK   SB2   ROUTE
RK  3000  0.0040   0.030           TRAP         20.0     1.0
```

Depending on the default routing option, the routing cards will be different. If the routing card option was selected when developing the draft file, the records will be at the bottom of the file. Move the routing cards to the correct location and clean up any trailing lines in the file as follows:

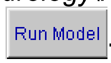



```
ID Project ID: KVLTEST1 - 10 Year Storm
ID
IT     1                  2000
IO     3
KK   SB1   BASIN
BA  .700
IN    15
PB 2.001
PC  .000    .009    .016    .025    .034    .042    .051    .059    .067    .075
PC  .087    .100    .119    .151    .234    .415    .763    .873    .915    .944
PC  .956    .967    .979    .989   1.000
LG  .240    .260   3.950    .540  95.000
UC 1.004    .456
UA     0       5      16      30      65      77      84      90      94      97
UA   100
KK   SB1   ROUTE
RK  3000  0.0040   0.030           TRAP         20.0     1.0
KK   SB2   BASIN
BA  .500
LG  .100    .290   3.350   1.020  80.000
UC  .771    .273
UA     0       5      16      30      65      77      84      90      94      97
UA   100
KK   SB2   ROUTE
RK  3000  0.0040   0.030           TRAP         20.0     1.0
ZZ
```

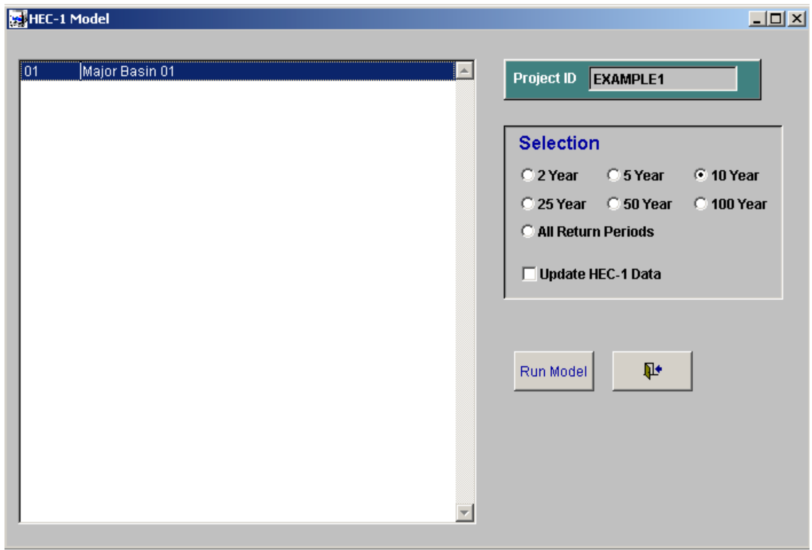
Finally after editing the ASCII file it can be imported. Click Import and select the appropriate file to import. A dialogue box comes up for the selection.



13. Run HEC-1 Model

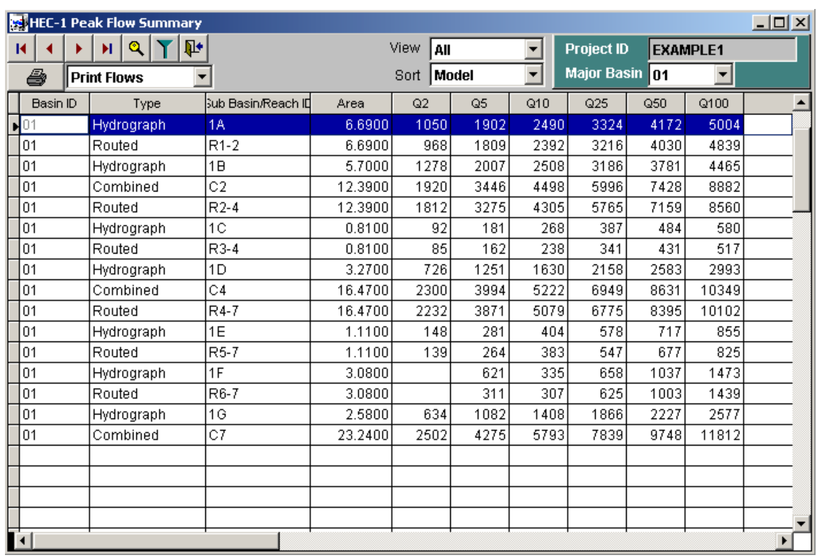
Select *Run HEC-1* from the *Hydrology\HEC-1* Menu. Select the appropriate Major Basin ID and Return Period and click 

.



14. View Model Summary Results

Select *View Summary Results* from the *Hydrology\HEC-1* Menu.



Basin ID	Type	Sub Basin/Reach ID	Area	Q2	Q5	Q10	Q25	Q50	Q100
01	Hydrograph	1A	6.6900	1050	1902	2490	3324	4172	5004
01	Routed	R1-2	6.6900	968	1809	2392	3216	4030	4839
01	Hydrograph	1B	5.7000	1278	2007	2508	3186	3781	4465
01	Combined	C2	12.3900	1920	3446	4498	5996	7428	8882
01	Routed	R2-4	12.3900	1812	3275	4305	5765	7159	8560
01	Hydrograph	1C	0.8100	92	181	268	387	484	580
01	Routed	R3-4	0.8100	85	162	238	341	431	517
01	Hydrograph	1D	3.2700	726	1251	1630	2158	2583	2993
01	Combined	C4	16.4700	2300	3994	5222	6949	8631	10349
01	Routed	R4-7	16.4700	2232	3871	5079	6775	8395	10102
01	Hydrograph	1E	1.1100	148	281	404	578	717	855
01	Routed	R5-7	1.1100	139	264	383	547	677	825
01	Hydrograph	1F	3.0800		621	335	658	1037	1473
01	Routed	R6-7	3.0800		311	307	625	1003	1439
01	Hydrograph	1G	2.5800	634	1082	1408	1866	2227	2577
01	Combined	C7	23.2400	2502	4275	5793	7839	9748	11812

Alternatively, the output file can be viewed in its original format by selecting *View Output File* from the *Hydrology\HEC Model*